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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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HISTORICAL NOTES ON ENGINEERS AND ENGINEERING ACTIVITIES IN CALIFORNIA

ADDRESS AT THE ANNUAL CONVENTION AT PASADENA, CALIFORNIA, JUNE 18, 1924

By C. E. Grunsky,* President, Am. Soc. C. E.

May I remind you that it is now 385 years since Ulloa concluded that there must be a great river discharging into the Gulf of California and that Lower California was not an island as had theretofore been generally assumed. A year later, that is, in 1540, Captain Hernando de Alarçon ascended the Colorado River to a point probably about 100 miles above Yuma, Ariz., and the same point was reached overland a little later the same year by Melchior Diaz who was detached from the expedition of Francisco Vasquez de Coronado then on a search for the famed seven cities oof Cibola.

These explorers found along and to the west of the Colorado River a dry forbidding country—long designated on geographers' maps as the Colorado Desert. The name, California, however, had already been attached to the larger unknown region extending far to the northwest. The name is from an old romance in which there is an account of an island, off somewhere from the Indies, which according to the story bore this name and which was inhabited by Amazons ruled by the queen, Califia.

Knowledge of the Pacific Coast region which is now embraced in the three States, California, Oregon, and Washington, was slowly acquired. The southern coast of California was explored in 1542 and 1543 by Juan Rodriquez Cabrillo. In 1579, Sir Francis Drake put into a harbor a short distance north of San Francisco to repair his ships and found the climate foggy, cold and forbidding. He called the country New Albion. In 1602 and 1603 Sebastian Viscaino discovered the Bays of San Diego and Monterey. Nothing, however, in the way of settlement came of these explorations and for more than 150 years thereafter the only inhabitants were the scattered Indian tribes.

The period from 1769 to 1823 brought the first marked change. During this time, under the leadership of Miguel José Serra (Junipero Serra), an able, kindly, far-seeing, zealous member of the Franciscan monks, twenty-one missions were established by this religious order, at which the spiritual and temporal well-being of the proselytes was looked after. It is at the sites of these missions that evidence of some early engineering activity can still be found. The Franciscans were no mean architects and planned their structures not only with pleasing exteriors and interiors, but also with proper regard for stability of walls, arches, and roof members. At some of these missions the

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irrigation canals which furthered intense cultivation of the soil can readily be traced. Presently, however, the support of the State was withdrawn. The secularization of the missions fell into the period from 1826 to 1840, and six years thereafter all the mission holdings had been sold to the Government.

This period (from about 1828 to 1845) was one of unrest. The population was sparse and scattered over a vast area. Dissatisfaction with Government officials found ready expression in protests against the actions of those in authority, and any small group of malcontents could cause serious disturbances. Thus, for example, Governor Victoria was deposed in 1831. Four years later Governor Chico was frightened out of the Province, and then Governor Gutierrez was driven out of office. In 1845 a like fate befell Governor Micheltorena.

However, there was now a steady increase of the foreign element. The Spanish prohibition against trading with others than Spaniards and against the admission of foreigners was constantly evaded. Most of the trading was done with Americans, and of the traders and sailors who came to the coast many established homes in the Province. Even the Russians had gained a foothold, having established a trading post at Bodega Bay in 1805, which they held until 1841. The Hudson Bay Company, too, extended its operations into the northern parts of California, beginning in 1830. Recognizing the menace of the increasing number of foreigners, Mexico tried to check their influx, but such efforts at restriction of immigration were frustrated by the local authorities who looked on such immigration with favor. As the foreign element consisted in the main of people from the Eastern States, it is not surprising to find that there were some who were early of the opinion that California should throw off allegiance to Mexico. There was a sporadic attempt to do this in June, 1846, which might have had unfortunate results, but which was followed so closely by the raising of the American flag at Monterey, on July 7, by Commodore John Drake Sloat, after the opening of the war with Mexico, that embarrassment was averted. After the Mexican War, by the treaty of Guadelupe Hidalgo, in 1848, California was ceded by Mexico to the United States.

Immigration from the Eastern States, mainly by the overland route, had already commenced in considerable numbers, when in January, 1848, the discovery of gold was announced. It took a few months for the news of this discovery to be generally accepted and then followed a rush to the gold mines, that is, to the rivers and creeks in the foothills of the Sierra Nevada Mountains, such as the world had never seen before; and with this great influx of humanity there arose at once a variety of engineering problems—water supply, sanitation, transportation, and methods of mining, stood out among these problems and resulted in many notable achievements. The early comers, of course, were thrown on their own resources.

Moreover, the thousands who gathered here—at the points of disembarkation, at the central stations from which supplies were forwarded to the mines, and along the mountain streams where the ground was being washed to relieve it of its gold—found themselves separated from the remainder of the world by the breadth of a continent. At first, only two alternative routes were available

for the shipment of goods, either the Panama route with precarious shipping facilities on the Chagres River and a poor road through tropical jungle, or that other route, uncertain as to time, around Cape Horn. It was some years before the third route by way of Lake Nicaragua became available, and it was not until 1855 that the Panama Railway was completed, and merchants and manufacturing establishments could begin to count on the arrival of their purchases of goods in the East on a reasonable schedule.

In the mild climate of California construction of buildings was, of course, at the outset, of the most temporary or makeshift character. For a business place at San Francisco, or in the interior, a light timber framework covered with canvas would suffice. The inside finish of even the better residences was paper on a cloth foundation. No high-class engineering attainment was required in the erection of such structures, but occasionally the man of vision and business daring, the man with the engineering type of mind, would see his way to fortune along some special line, as, for example, a certain new arrival who recognized the special value of tacks as a building material. He first quietly ascertained what stores at San Francisco included tacks among their wares. Having located every stock thereof in town, he went, in turn, from one business house to the other, in each case inquiring whether they had tacks to sell. How large was their stock? What was the price? And then he would take the whole stock with the statement that he would be in a little later to arrange for its delivery. Each merchant presently found that customers coming in for tacks could not be supplied and the neighboring store which in the past had always been able to help out, could be of no assistance because it, too, had just sold its entire stock of tacks. When the shrewd purchaser of tacks made his second round he had no trouble in selling the tacks back to their original owners at a handsome profit. Tacks about this time, as we are told in the records of that period, were worth their weight in gold.

Such speculations involving a cornering of the market were not infrequent. Quite apart from these spectacular transactions, however, it was a matter of great concern to have goods arrive on a depleted market before the rivals in business were supplied. A few hours of advance information as to the arrival of a ship known to be bringing a certain cargo was, therefore, at times of great value to the merchant. It was for this reason, and to give early notice of the arrival of mail, that a signal station was established on Telegraph Hill at San Francisco from which by the aid of semaphores notice was given to the Montgomery and Sansome Street merchants of the type of each vessel that entered Golden Gate.

How different in this day of the telegraph, telephone, and radio! This condition of isolation, not fully broken even by the completion of the transcontinental railroad in 1869, is to be taken into account when considering the achievements of pioneer engineers. Their problems had to be solved without the aid of the expert from the East or from abroad and at times with facilities which we, to-day, would call quite inadequate to the task.

Each miner, for example, relied on his own ingenuity to overcome obstacles and to work his claim to the best advantage. What the placer miner needed above all else was water with which to wash away the dirt, sand, and gravel

from the grains of gold that he was after. Generally, he knew where the water was, but not always was it within his range of skill to select the best route for bringing it from the source to his mine. In those days, that is, early in the Fifties, whoever could find the difference in elevation between two points was a person of special attainments and was rated as an engineer. The available fall was all the miner wanted to know. As to dams and ditches, flumes and sluice-boxes, he could take care of that himself. The demand in such circumstances on the engineer was so limited that not a few who could barely read a compass went into the business or profession of engineering although often but poorly equipped for the service. In this connection the story may be recalled of the two young men who at one of the mining camps on the Stanislaus River undertook to construct, during the dry months of summer, a ditch that was to supply the miners with the run-off of a tributary area trapped in a small reservoir near the head of the creek. A "go-devil" was used in making the survey. This, as some of you may know, consisted of a horizontal member, a rod long, with a vertical plumb-bob attachment to control the horizontal position. The fall from stake to stake was secured by placing a chip under one end of the "go-devil". All went well as far as construction was concerned, and at the camp the miners piled up the dirt at their claims, ready to be washed when the water arrived. At last the rains came and the water began to accumulate in the reservoir. The next morning the gates were to be opened. As a precaution, however, the young men went up after dark to try out the gates and to see how the ditch would function. To their consternation they found that the water would not flow in the ditch. Too late they realized that they had made their survey with the chip under the wrong end of the "go-devil". Silently they packed their belongings and were far away when the next day the miners awoke to their disappointment.

Some of the difficulties connected with placer mining, however, were more happily solved. It was soon noted that if water could be brought under pressure to the bank which was to be excavated, much labor could be saved and vastly greater quantities of material could be delivered into the sluice-boxes. Stove-pipe, wooden boxes banded with iron, and canvas hose up to 8 in. or more in diameter could be seen in use, here and there, to conserve the head and throw the water against the bank with some force.

We are told, for example, that as early as 1852 at Yankee Jim's, in Placer County, a miner delivered the head of water into a barrel, set about 40 ft. higher than his mining ground, and conveyed it from this barrel in a rawhide hose, 6 in. in diameter, to the gravel to be washed. At the end of the hose was a 4-ft. length of tin tube, fitted with a 1-in. nozzle. About the same time, at Buckeye Hill, near Novada City, another miner, named Chabot, allowed the water to collect in a sort of wooden penstock which fed into a set of wooden boxes strengthened with iron clamps so that they could withstand a pressure of 50 or 60 ft. A 4-in. canvas hose was attached to the lower end of these boxes.

Quickly, then, followed the development of the hydraulic nozzle—the Monitor, the Chief, the Dictator, and the Giant—with which powerful jets were turned against the gold-bearing layers of earth, gravel, and conglomerate.

Whole mountains were sluiced away to get at and work these gold-bearing layers. The art of hydraulic mining was rapidly developed, and many of those who were most active in its development are well remembered as leading mining and hydraulic engineers of this period. We may name in this connection the late A. J. Bowie and Hamilton Smith, Members, Am. Soc. C. E., and Messrs. Gardner F. Williams, Hennen Jennings, Cleveland Perkins, and A. H. Brigham, all expert hydraulicians, of whom only one or two now survive.

Not only were these engineers particularly concerned with the new type of mining, but to them came the vexing problem of the flow of water in pipes and the design of devices for the measurement of water. Both Bowie and Hamilton Smith have left us the results of some of their studies and experiments in volumes that were classic in their day.

The crossing of a deep valley with a water conduit at the Cherokee Mine was a problem of a new type. Heading the valley by ditch and flume on grade was out of the question, neither could there be any trestle support for a direct crossing, as the depth of the ravine below grade was about 600 ft. In 1869, an inverted siphon was installed, 30 in. in diameter and 4 500 ft. long, constructed of riveted iron pipe—an unheard of proceeding. How could the thin shell of iron be depended on to withstand the stress to which it would be subjected? How would temperature changes affect such a pipe? How was its collapse to be prevented in case of sudden release of its contents? Despite adverse criticism, however, the pipe line was a success. It not only fulfilled its purpose admirably, but it demonstrated that iron, suitably protected by a coating of tar and asphalt, could be relied on for long-time service under high pressure.

The construction of this pipe line under high pressure was followed three years later by a still bolder installation at Virginia City, Nev., where the late Hermann F. A. Schussler, M. Am. Soc. C. E., who was best known as the Chief Engineer of the Spring Valley Water Company which supplies San Francisco with water, had the courage to construct a 12-in. pipe line across the head of the Washoe Valley. This pipe line was 7 miles long and its lowest section was under a head of 1 720 ft.

The serviceability of thin plates of wrought iron and of steel for use in the building of water conduits under high pressure was established by these and other similar installations, and a valuable contribution to the art of engineering was thus made.

A little later the owners of the famed Comstock Mines were in trouble because of the lack of pumping facilities to unwater the deep workings. A first-class pumping plant was required. Who could furnish it? The late Mr. George W. Dickie, who was afterward Manager and Chief Engineer of the Union Iron Works, and the builder of the U. S. S. *Oregon,* and other engineers talked the matter over and decided that the plant could be constructed if only a suitable engine shaft could be obtained. There were no facilities for making one on this coast at that time. Some one remembered the wreck of a steamer that had met with disaster in the Sacramento River near Sacramento a year or two before. Why not purchase the wreck and recover her shaft? This was

done, and the great Comstock pumps were designed, built, and assembled about this shaft, and were soon installed and in successful operation.

The movement of water follows certain inexorable laws and when confined, the imparted energy may work havoc if suitable opportunity is not provided for its release. Thus, in this case, rhythmic pulsations were set up in the vertical discharge pipes, which resulted in water-hammer and pressures at certain points of the pipes, of destructive force. Even gun-barrel types of pipes were ruptured. The studies of the water-hammer phenomena of the Comstock pump were undertaken by the late W. R. Eckart, M. Am. Soc. C. E., who designed and used most delicate and intricate devices for the determination of the pressure nodes in the pipes and of the frequency and rate of travel of the pulsations. Among other devices, a tuning-fork was used to trace a wavy line on a strip of moving paper, in order to facilitate the precise determination of time intervals. The facts ascertained were made the basis of corrective measures which remedied the trouble.

Many years later, again on the Comstock Lode, the unwatering problem of lower levels was once more to the fore. Specially designed ejectors were installed to lift water about 400 ft., to the level of the Sutro Tunnel. The impelling force was supplied by water from the surface of the ground, discharged through the nozzle of the ejector under a head of more than 2 000 ft. Here was another evidence of the resourcefulness of the Western engineer in overcoming difficulties.

He has been active, too, along other lines, and has accomplished many notable results. High-head water-power installations, for example, were revolutionized when means were found to utilize water impact to advantage. The jet from a nozzle impinging on the paddles of a wheel of fairly large diameter is a simple installation and fairly efficient.

The requirements for power in the gold mining regions of California contributed much to bring the impact water-wheel to its present state of perfection. Water, as a rule, was not available to the California miner in abundance. It could frequently be used, however, under a high head. With very imperfect knowledge of the general principles governing the generation of power in tangential wheels, the miners of California evolved the "hurdy-gurdy" water-wheel, which was quite well developed as early as 1854. This wheel had small paddles, blocks of wood, about 8 in. apart, on the periphery of a wheel, against which there was discharged a jet of water from a nozzle under high head. The wheel was of wood and its hardwood shaft had oak bearings. An efficiency of 40% is reported to have been secured by this crude device. In 1866, a cast-iron impact wheel was made and proved so successful that the "hurdy-gurdy" was doomed. About 1870, curved buckets and, a few years later, a flat instead of a round jet were made features of what became known as the Knight wheel, after its inventor, Mr. S. N. Knight. Within the next eight years several investigators and builders of water-wheels hit on the idea of splitting the jet in the bucket with a sharp-edged wedge, thus creating a double cup or bucket. The outcome of this was the well-known Pelton wheel named after Mr. L. A. Pelton, one of those who claimed priority of invention. A patent was issued to him in 1880. As a result of progressive improvements,

California still leads the world in the degree of perfection attained in the impact type of water-wheel.

In the field of gas-making the California engineer has not lagged behind. In fact, he has given to the world processes which have revolutionized the art of converting crude oil into gas. The leader along this line has been Mr. E. C. Jones, who is still active as a Consulting Engineer of the Pacific Gas and Electric Company. His process of manufacturing gas from oil was a great step forward. Then came the improved process about 12 years ago, by means of which the formation, as theretofore, of large quantities of lampblack—a by-product—was eliminated and the gas output was materially increased. This was a most notable achievement.

Marked progress has been made in California, too, in the design of certain types of dredges adapted to special local requirements. In the delta of the San Joaquin and Sacramento Rivers there were originally about 500 sq. miles of low-lying lands, subject to submergence at high tide and well covered with water whenever the rivers were in flood. These lands and other overflow areas, farther up stream, have been reclaimed in the course of time and made arable by protecting them against overflow with embankments frequently 15 to 20 ft. high. In many cases, these embankments were placed on supporting ground which was compressible under the weight of the superimposed levee. Due to the settlement of such foundations and to the shrinkage of much of the material placed in the embankment and also to occasional sloughing off of long stretches of levee, vastly more material had to be handled in building the levees, than is now represented by their volumetric contents. Engineers on this work were early confronted with the problem of designing machines that could build levees efficiently. One of the revolutionary types of dredges was the suction dredge, of which probably the earliest of its kind was built and put into commission by Mr. A. W. von Schmidt, one of the best known engineers in California in his day. It was designed by his brother, Mr. Julius von Schmidt.

The first dredge of this type was intended to suck up sand from the river bed through pipes and deliver it to the shore where it was to fill the space between two dikes built up by hand of blocks of peat. A centrifugal pump was used to draw water and an inverted cup-shaped hood over the end of the suction pipe lying on the sand of the river bed forced the inflowing water to carry with it a certain quantity of sand. It was soon found that there could be no control over the quantity of sand picked up by the water going under the edges of the hood. Sometimes, the water was well charged with sand and, again, it came clear without any sand. Some stirrer or cutting device must be provided. Experiment after experiment was made and others, too, entered the field and perfected devices to overcome mechanical difficulties here and there. von Schmidt's most notable competitor was Mr. A. B. Bowers, who in the course of time controlled most of the important patents connected with hydraulic or suction dredges. From this beginning in California, the art has advanced until now the suction dredge and, particularly, the ocean-going suction dredge, has become an indispensable tool of the harbor engineer.

Not only in the matter of suction dredging, however, have the achievements of California engineers been notable—the clam-shell long-boom dredge, capable

of dropping its load well inshore away from the unstable river bank, has here found its highest development. No sooner were dredges built with booms 100 to 120 ft. long, with 2-yd. buckets, than others were planned with booms 150 ft. and more in length, with buckets of 5 and 6 cu. yd. capacity. One difficulty after another of operation with long booms was overcome until now there are in successful use dredges of this type with booms more than 200 ft. long, and, at least, one with a boom about 240 ft. long.

What has just been said of the long-boom clam-shell dredge is equally true of the chain-bucket gold dredge. This type of dredge, or rather its prototype, was brought to California from New Zealand about 25 years ago by Mr. R. H. Postelthwaite. Hardly had the first small dredge been built and put in operation before dredges of greater capacity were demanded. The development of the large type of machine now in use, costing nearly $500 000 and handling in excess of 10 000 cu. yd. of material per day, quickly followed. Not only is this the case, but the original method of burying the soil that is worked over, under ridges of barren, unsightly cobble-stones, has been modified so that the ponded machine now builds up in its rear a bank in which there is a fair quantity of soil at the surface, requiring but little leveling to make it available for agricultural purposes. The great economic waste of early operations which ruined, for all time, a considerable area of agricultural land is thus avoided.

It is about 40 years ago that Mr. F. E. Brown astonished the Engineering Profession by his bold construction of a granite masonry arched dam at Bear Valley in the San Bernardino Mountains, of such slender dimensions as to cause not a few predictions of early failure. This dam has now been superseded by a higher multiple-arch dam. Its curvature conformed to the arc of a circle with a radius of 337 ft. At 45 ft. below its crest height, its thickness was 8 ft. It is understood never to have given any trouble. Its success went far toward securing recognition of the dependability of the horizontal arch. Dams of the arch type are now numerous.

In connection with the building of dams some reference should be made to the achievements of the late James D. Schuyler, M. Am. Soc. C. E., a California engineer, who for many years was recognized as one of the foremost authorities in the world on the building of dams. It fell to him to add to the height of the Sweetwater Dam, a concrete structure, near San Diego. All difficulties in connection with this work were successfully overcome, and the narrative of the successful completion was published in such an attractive and enlightening setting that the author found himself famous as a dam builder with commissions pouring in from near and far. Although Mr. Schuyler was a native of New York, he had early been attracted to the frontier by railroad work, drifting into California in the Seventies from somewhere in Colorado. Ever since he has been claimed as a Californian.

Reference has already been made to the development of the method of hydraulic mining in California. It remained for California engineers to recognize that the deposit of water-borne material could be controlled to such an extent as to be made serviceable in the building of earth dams. This principle was applied on a large scale for the first time, it is believed, in the

erection of the San Leandro Dam near Oakland (Chabot Reservoir), about 1880, perhaps somewhat earlier. The method has been fully developed by such eminent engineers as the late Mr. Schuyler and J. M. Howells and W. H. Mulholland, Members, Am. Soc. C. E., the latter of whom is Engineer of the Los Angeles Aqueduct Board, and others. The method, although not applicable in the construction of every earth dam, is nevertheless now accepted, the world over, as one of the standard methods of construction.

The arched masonry (preferably concrete) type of dam, previously referred to, was the forerunner of the multiple-arch dam, which is believed to have reached earliest recognition, as a dependable type of structure, in California. The first notable dam of this type in California was built at Hume, by Mr. John S. Eastwood, for the Hume Lumber Company, and other dams soon followed. One of the great advantages of this type of structure is the fact that stresses are susceptible of quite definite computation and that, therefore, no surplus material is required in its construction. Multiple-arch dams are now in use up to 150 ft. in height, and no reason is apparent why this type of structure should not be used for even greater heights.

The first cable street-car system used anywhere was on Clay Street, San Francisco, about 1878. Some means had to be found to raise the street cars on a direct course to the hilltops of San Francisco. Propulsion by gripping an endless traveling cable was suggested and favorably considered by the promoter and builder of the first cable railroads in San Francisco, Mr. A. S. Hallidie. He was assisted in solving mechanical problems by Mr. William Eppelsheimer, who took out a number of patents on the resulting devices. All the difficulties connected with this method of propelling street cars were gradually overcome by the resourcefulness of California engineers among whom Mr. Henry Root is particularly notable. The use of the cable system was extended to all parts of the world, maintaining itself until displaced in recent times, except on routes with very steep inclines, by the more modern electric railway systems.

California has repeatedly astonished the world with boldness of installation for long-distance transmission of electric energy. The first worth-while long-distance installations were made in California, which has remained in the lead in the matter of distance of transmission and of high voltage used on transmission lines. On this subject, in a paper read before the Technical Society of the Pacific Coast on February 6, 1885, E. J. Molera, M. Am. Soc. C. E., stated:

"The most important problem now presented to electricians is the question of the transmission of power to great distances. * * * Several years ago, Professor Thomson of Glasgow announced that with a copper wire ½ in. in diameter 27 000 h.p. could be transmitted from Niagara Falls to New York with a loss of only 7 000 h.p."

In 1882, with facilities provided by the Munich Electrical Exposition, Marcel Desprez "succeeded in transmitting ½ h.p. from Wiesbaden to Munich a distance of 59 km. (37 miles) through an ordinary telegraph wire with a loss of 62 per cent." Other experiments were made by him in the following years with like results. Commenting on these experiments, Mr. Molera said:

"The main difficulty lies in overcoming the resistance of the electric conductor, which is in proportion to the distance and inverse to the square of its section. To overcome it by increasing the diameter of the conductor is not possible on account of the expense. The other way is by generating electricity of a high electro-motive force, which is a very easy matter."

He then expressed the opinion that the difficulty of effecting insulation at the pole supports is more apparent than real, and concluded with the statement that nothing along the line of long-distance transmission had yet been done in the United States.

Single-phase generators and short-distance transmissions were installed at various places in the United States from 1889 to 1893. Probably the first notable electric plant, one with fairly long transmission of power, was that at Portland, Ore., where the transmission line had a length of 13 miles. This was built in 1889. Among the other single-phase plants in California of this period was one at San Antonio (1891). Here, 150 h.p. was transmitted 15 miles. The plant at Bodie (1893) had the same capacity, transmitting over a line 13 miles long.

An experimental line, 110 miles long, between Lauffen, on the River Rhine in Germany, and Frankfort, gave the first successful demonstration in 1891 of the transmission of a three-phase current. Operations over this line from turbine shaft to lamps at a voltage of about 16 000 was at an efficiency of 74 to 75 per cent.

Two years later, the Redlands, Calif., plant of 250 kw. was completed, the transmission of the three-phase current being over a line 23 miles in length, at 11 000 volts. The Folsom-Sacramento plant, the completion of which was quite an event in 1895, was built to transmit energy over 22 miles of line, also at 11 000 volts. The first actual high-head power plant was built by the San Joaquin Light and Power Company in the mountains northeast from Fresno. The total available head at this plant is 1 420 ft.

In the matter of transmitting electric energy at ever-increasing voltage, California has kept abreast of the remainder of the world, leading at present with the Southern California Edison Company's transmission line from Big Creek to Los Angeles and the Pit River transmission line of the Pacific Gas and Electric Company, both of which are operated at 220 000 volts.

As an illustration of how Pacific Coast engineers of 40 or 50 years ago managed, let me quote from the Presidential Address of the late Mr. George W. Dickie, delivered December 16, 1905, before the Technical Society of the Pacific Coast. Mr. Dickie, after stating that, on his arrival from Scotland in 1870, he found little activity at San Francisco in his special line of marine engineering, said:

"Reading the newspaper one day, my eye caught an advertisement wanting a competent man to do the planning and erecting of a gas-works plant—apply to the Risdon Iron Works. I looked hard at this advertisement; a competent man was wanted and I considered myself competent, but this man was wanted to plan and erect gas works and I knew little or nothing about gas works. Nature had endowed me with the faculty of using a little gas to help myself when such help was needed, but in this case I had not enough knowledge to

warrant its expanding to the required bulk. This, however, was the only thing I had seen for any man to plan in the months I had been here. I wrote to the Risdon Iron Works saying that I would like to undertake the work for which they wanted a competent man."

Mr. Dickie then read up at a technical library on the subject of gas-works and when he met the engineer who represented the promoter of the gas-works project, easily impressed him with his superior knowledge on the building of gas plants. He was employed and the works were built and put into operation without any trouble. Mr. Dickie continuing the narrative said:

"About this time, when I was at the height of my fame as a gas engineer, the Pacific Mail Steamship Company concluded to fit one of their side-wheel steamers with a surface condenser. One day I overheard their Superintending Engineer talking the matter over with Mr. Moore (the Superintendent of the Risdon Iron Works) and wondering where they might get hold of some one who had had some experience in building surface condensers to take charge of this work; so as soon as Mr. Moore was alone I suggested to him that he put this matter in my hands and I would see it properly carried out. 'No', said he, 'that would never do. You are a gas engineer, and this needs a marine engineer who knows all about surface condensers.' As my ambition could not be reached by the way of gas engineering I had to confess the trick that I had played on him in the matter of gas, and produced my letters of recommendation from engineers and shipbuilders in Scotland * * * and finally succeeded in getting a start on the road that I have trudged along ever since."

And, again, quoting Mr. Dickie from the same address, in referring to the unwatering of the Comstock Mines about 1874, after noting some of the adverse conditions as, for example, working where the heat was so great as hardly to allow life and where a drop of water raised a blister on the skin, he said:

"Many methods were put in operation in this out-of-the-way corner of the world, when there was not time to make records, which have been re-invented since, such as compressing air in two stages and the reheating of air during expansion while doing work. Much original work in hydraulics resulted in attempts to drain the lower levels of the Cholar, Norcross, and Savage Mines through their combination shaft; all this work was designed by me without any precedent to go by, and feeling one's way under the conditions that prevailed in these mines was an education in how to control one's nerves if nothing else. On the 2 400-ft. level running into the old Savage workings we had a stone wall built across the drift, with a pipe and a valve on it to control the flow. When things went wrong and the valve had to be closed the water would rise behind that wall till it stood at 1 700 ft. [700-ft. head]. I often looked at the pressure gauge and imagined how it would be if that wall let go."

As already intimated, the late Mr. W. R. Eckart was closely associated with the work on the Comstock. This was referred to by Mr. Dickie when he said:

"I have often talked with Mr. Eckart about it [referring to making a record of the engineering experiences on the Comstock] and I think that he has more notes of what was done than any one else, but he has not seen his way to use the information he has. * * * I have never forgiven myself for not recording all that came within my own observation as it occurred, as memory plays many tricks upon us if we try to drive her back 30 years or so, and I will not risk the danger of her leading me astray."

Among the older engineers of California not already mentioned who attained high standing and whose records the profession may well be proud of, the following remain to be mentioned. There was, for example, Mr. Calvin

Brown, the first Chief Engineer of the Spring Valley Water-Works, San Francisco, who later designed and built the first dry dock at Mare Island; the late Theodore DeH. Judah, M. Am. Soc. C. E., who made surveys for the transcontinental railroad across the Sierra Nevada Mountains; the recently retired Chief Engineer of the Southern Pacific Company, William Hood, M. Am. Soc. C. E.; Gen. B. S. Alexander, and a little later, the late Col. G. H. Mendell, M. Am. Soc. C. E., who were, in turn, at the head of the Pacific Coast contingent of the Engineer Corps of the U. S. Army, and were consulting engineers on numerous investigations and construction enterprises; the late George Davidson, Hon. M. Am. Soc. C. E., for many years at the head of the Pacific Coast Division of the U. S. Coast and Geodetic Survey, whose remarkable work in the triangulation system, which connects the West with the East, was alone sufficient to establish an enviable reputation. Among the Mining Engineers of California other than those already named, who have achieved international reputation there should be mentioned Mr. Charles F. Hoffman; the late Luther Wagoner, M. Am. Soc. C. E., whose paper on stresses in arched dams, in collaboration with Mr. Hubert Vischer, was a notable contribution to engineering knowledge; Mr. Phil Deidesheimer who invented the square-frame method of mine timberings first used in the Comstock, and some half dozen more whose records are not yet complete, but some of whom have made a most profound impress on the affairs of the world and the progress of civilization. It will suffice to add the names of Messrs. Ross E. Browne, John Hays Hammond, Charles Butters, Hans C. Behr, and the Hon. Herbert Hoover, M. Am. Soc. C. E.

The work of Mr. J. C. H. Stut contributed materially to the success of the cable railways. Messrs. John Richards and Bryon Jackson, the latter a manufacturer who never claimed rank as an engineer, will long be remembered for their achievements in spreading the use of and improving the centrifugal pump, so, too, Professor G. E. Hesse who was for many years at the University of California, and whose studies developed the principles of the impact water motor and laid the foundation for the full development of the Pelton water-wheel; Mr. Walter James who pioneered the irrigation work on the Kern River, in Kern County, which served for many years as a standard to be closely followed where conditions permitted; William Hammond Hall, M. Am. Soc. C. E., who laid out Golden Gate Park in San Francisco and who, as State Engineer of California, inaugurated the study of the water resources of this State and outlined the features of the law subsequently enacted under which irrigation districts could be organized; and, of course, the many engineers of the younger set, whose achievements although not always so unique and striking are, nevertheless, frequently quite as meritorious.

As this Society has on other occasions been told, we are not given to perpetuate the memory of our pioneer engineers in stone or bronze. It is fitting, therefore, that some attention as in this inadequate attempt, be given to their achievements, and that their names be recalled as occasion offers. May what they have so well done be a source of encouragement and inspiration to those who to-day are carrying on like work and who are adding to the massiveness of the foundation of knowledge, experience, and accomplishment, from which future generations may soar to heights not yet dreamed of.

THE DESIGN OF A MULTIPLE-ARCH SYSTEM AND PERMISSIBLE SIMPLIFICATIONS

By A. C. Janni,* M. Am. Soc. C. E.

To Be Presented September 3, 1924.

Synopsis

A strictly theoretical design of a multiple-arch system the arches of which cannot be considered as having fixed ends, is hardly practical. It is the aim of this paper to show whether, with such a system, it is possible, for practical purposes, to simplify or shorten the strictly theoretical method. From the results obtained, it seems that some simplifications are quite permissible.

General

The design of an elastic arch, no matter whether analytical or graphical, or a combination of both, represents one of the most complex problems in engineering. Formerly, relatively few arches were built, but, recently, especially since the advent of reinforced concrete as a material for construction, the rapid increase of road building has necessitated more bridges and the arch span has become the type most generally used. Often, due to physical conditions, a series of arches is required, in which case, the piers supporting this system of arches may be of such dimensions that they cannot correctly be assumed as unyielding under the action of loads.

The case in which an arch may be considered as having unyielding supports—as with a single span founded on rock, or with an arch system, using piers of such dimensions that each arch may be considered as having fixed ends—has been treated by theory in several ways; in recent years, there has been added another method, purely graphical,† that is founded exclusively on the theory of the Ellipse of Elasticity.

This same theory also lends itself to the more complex problem of an arch the supports of which are yielding. It will be applied to a practical case, in which three arches, A_1, A_2, and A_3, have been considered as supported by two rather slender piers, P_1 and P_2 (Plate II).

For the sake of clearness, this problem has been treated, first, by considering A_2, one of the three arches, as having fixed ends, making the design accordingly, and obtaining the influence lines of the moments; and, second, by considering Arch A_2 as being between two yielding supports, the one on the left being the combination, A_1, P_1, the second being A_3, P_2. The influence

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in December, 1924, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

* Cons. Engr., New York, N. Y.

† "Reinforced Concrete Construction," by George A. Hool, Vol. III, Chapter 8.

lines, in this case, have been computed for the three arches, and have been plotted, together with the influence lines obtained in the first case, so that the difference of moments affecting the same sections under both assumptions may be seen at a glance.

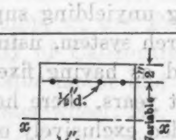
Also, for the sake of clearness, Plates II, III, and IV are shown in light and heavy lines; the light lines refer to the case of Arch A_2 on unyielding supports; and the heavy lines refer to the assumption of Arch A_2 on yielding supports, and to all variations of moments and stresses acting on this system and arising from the assumption made.

Design Assuming Unyielding Supports

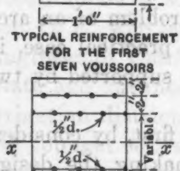
Plate II shows half an arch and its corresponding supporting pier. For convenience and clearness, a summary of the procedure will first be given under the assumption that the arch has fixed ends.

The geometrical axis has been divided into ten equal parts, each part being 2.587 ft. long, and the center of each voussoir has been located. The semi-axis of the ellipse of elasticity of each voussoir, along its geometrical axis, has been computed by the formula: Major semi-axis $= \rho = \sqrt{\frac{l^2}{12}} = 0.748$ ft., l being the length of the voussoir. Then, the moment of inertia, I, about the horizontal gravity axis, the area, A_1, A_2, etc., and the minor semi-axis of the ellipse of elasticity, P_1', P_2', etc., were computed for each voussoir. The results are given in Table 1.

TABLE 1.—Elements of Arch Sections.



TYPICAL REINFORCEMENT FOR THE FIRST SEVEN VOUSSOIRS



TYPICAL REINFORCEMENT FOR THE LAST THREE VOUSSOIRS

I, in feet.[4]	Area, in square feet.	ρ', in feet.
$I_1 = 0.101$	$A_1 = 1.162$	$\rho_1' = 0.294$
$I_2 = 0.107$	$A_2 = 1.182$	$\rho_2' = 0.300$
$I_3 = 0.113$	$A_3 = 1.202$	$\rho_3' = 0.306$
$I_4 = 0.120$	$A_4 = 1.222$	$\rho_4' = 0.313$
$I_5 = 0.127$	$A_5 = 1.242$	$\rho_5' = 0.319$
$I_6 = 0.134$	$A_6 = 1.262$	$\rho_6' = 0.326$
$I_7 = 0.150$	$A_7 = 1.302$	$\rho_7' = 0.339$
$I_8 = 0.187$	$A_8 = 1.524$	$\rho_8' = 0.350$
$I_9 = 0.217$	$A_9 = 1.584$	$\rho_9' = 0.370$
$I_{10} = 0.249$	$A_{10} = 1.644$	$\rho_{10}' = 0.389$

From the quantities computed in Table 1, the elastic weights of the voussoirs were found, as shown in Table 2.

PLATE II.
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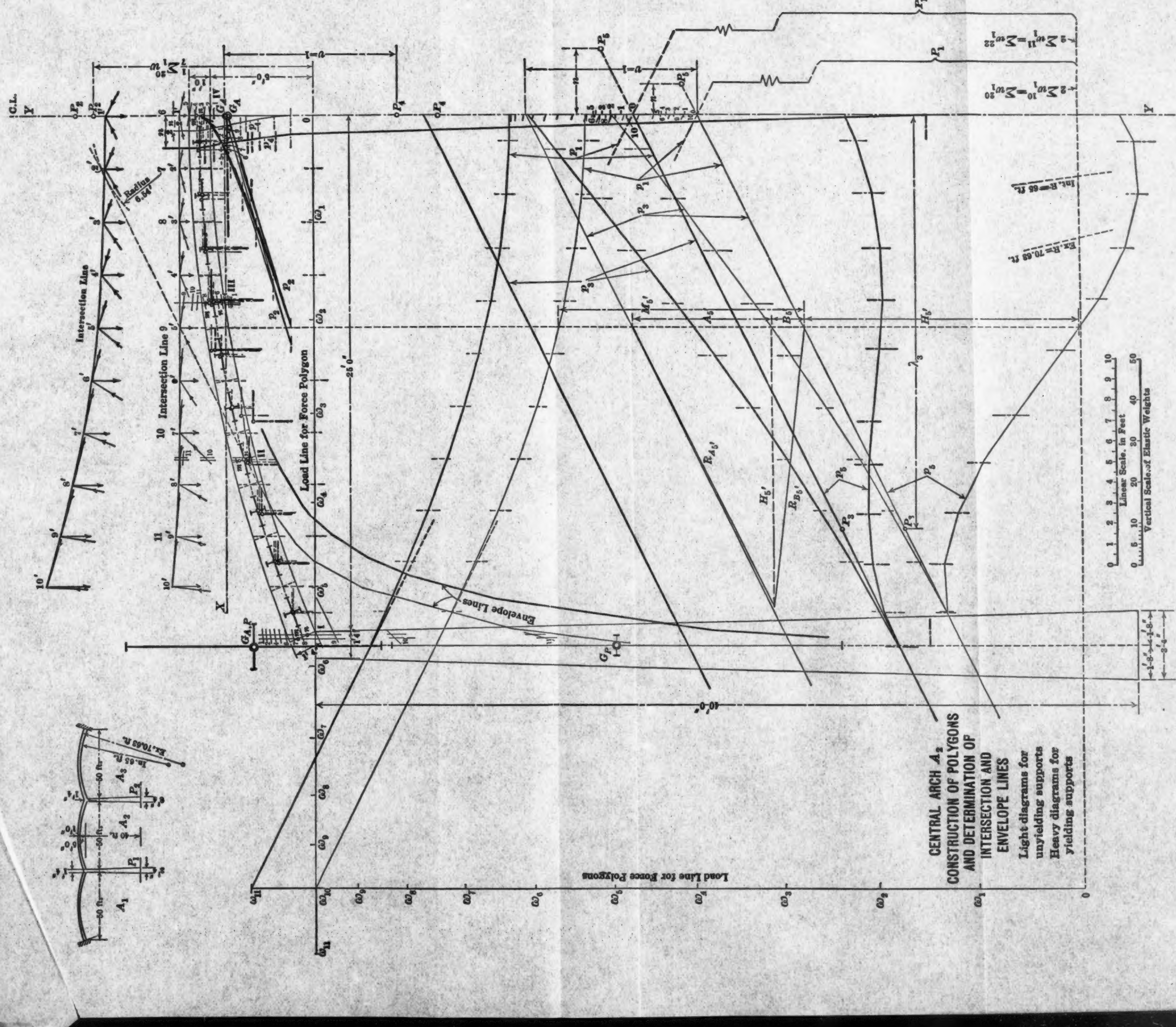

CENTRAL ARCH A_2
CONSTRUCTION OF POLYGONS AND DETERMINATION OF INTERSECTION AND ENVELOPE LINES
Light diagrams for unyielding supports
Heavy diagrams for yielding supports

PLATE
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JANNI
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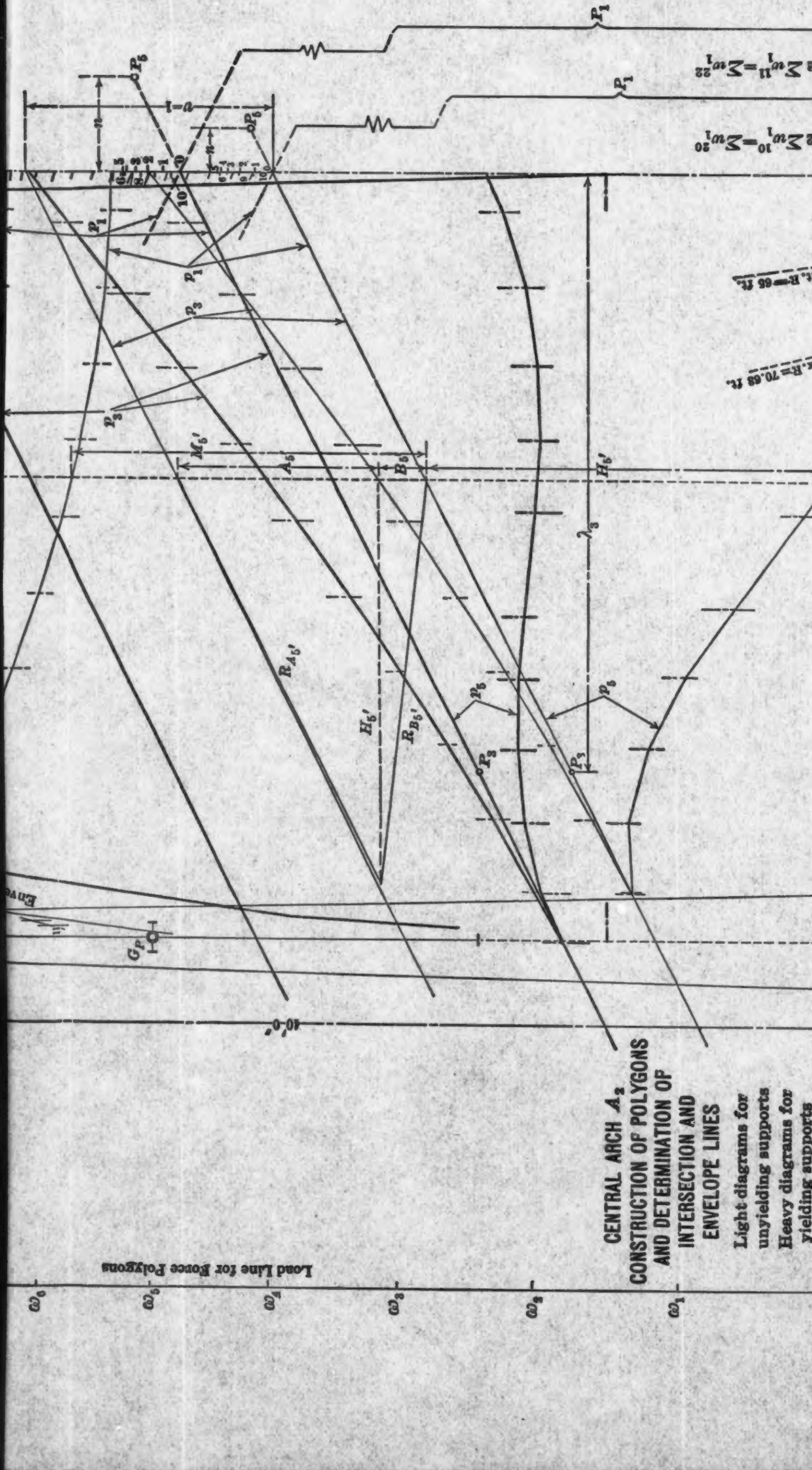


PLATE II.
PAPERS, AM. SOC. C. E.
AUGUST, 1924
JANNI ON
OF A MULTIPLE-ARCH SYSTEM

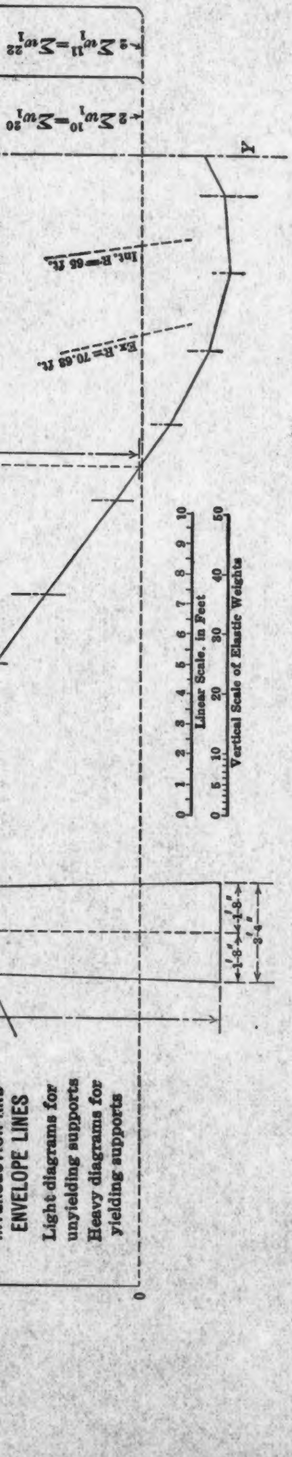


TABLE 2.—ELASTIC WEIGHTS OF ARCH SECTIONS.

For $E = 1.00$:	*(Brought forward)* $= 114.29$
$\frac{\Delta s}{E I_1} = \frac{2.58}{0.101} = 25.54$	$\frac{\Delta s}{E I_6} = \frac{2.58}{0.134} = 19.25$
$\frac{\Delta s}{E I_2} = \frac{2.58}{0.107} = 24.11$	$\frac{\Delta s}{E I_7} = \frac{2.58}{0.150} = 17.20$
$\frac{\Delta s}{E I_3} = \frac{2.58}{0.113} = 22.83$	$\frac{\Delta s}{E I_8} = \frac{2.58}{0.187} = 13.79$
$\frac{\Delta s}{E I_4} = \frac{2.58}{0.120} = 21.50$	$\frac{\Delta s}{E I_9} = \frac{2.58}{0.217} = 11.88$
$\frac{\Delta s}{E I_5} = \frac{2.58}{0.127} = 20.31$	$\frac{\Delta s}{E I_{10}} = \frac{2.58}{0.249} = 10.36$
Total	186.77

Polygon p_1.—The elastic weights were laid off, at a convenient scale, vertically in order, as shown on Plate II, and a pole distance assumed equal to twice the total sum of these elastic weights, giving the polygon of forces with a pole, P_1. Verticals through the centers of gravity of the ten voussoirs were drawn, and, considering each elastic weight as a force applied vertically, at the center of gravity of the voussoir, the funicular polygon was drawn.

It is to be noted, once for all, that the scale for the elastic weights, as shown on Plate II, is quite independent of the scale of the drawing.

Any vertical force applied to the arch will induce a moment equal to the product of the force and the ordinate that this polygon intercepts on the line of the applied force. For this reason, Polygon p_1 is called the Polygon of Moments.

Polygon p_2.—The same elastic weights were laid off in order horizontally, and a pole distance, P_2, taken. This pole distance, for convenience (as will be seen later) was taken as an aliquot part, ξ $\left(\text{assumed as } \frac{1}{7}\right)$ of $2\, \Sigma_1^{10}\, w$, that is:

$$\xi \,.\, 2\, \Sigma_1^{10}\, w = \xi \,.\, \Sigma_1^{20}\, w = \frac{1}{7}\, \Sigma_1^{20}\, w.$$

Horizontal lines passing through the centers of gravity of the voussoirs were drawn, and assuming that these horizontal lines are lines of action of the elastic weights now acting horizontally, the funicular polygon, p_2, was constructed, corresponding to the polygon of forces, P_2. The last side of Polygon p_2 cuts the vertical axis of symmetry, $Y\,Y$, of the arch at the center of gravity, G_A, of the system of elastic weights considered as acting at the appropriate centers of gravity.

Polygon p_3.—The next step is to locate the anti-pole* for the ellipse of elasticity of each voussoir with respect to the vertical axis of symmetry, $Y\,Y$, of the arch. In Plate II, these anti-poles have been located for the first six voussoirs; for the remaining four voussoirs, they would practically coincide

* See Appendix, Paragraph 7, *et seq.*

with the center of the corresponding ellipse, and are so considered. The statical moments of the elastic weights, applied vertically at the centers of the voussoirs, the amount of each being found by extending the sides of Polygon p_1 to the axis, *Y Y*, have been considered as forces applied vertically to the anti-poles found previously. With a pole, P_3, at an arbitrary distance from the axis, *Y Y*, a corresponding funicular polygon, p_3, was drawn.

This polygon, p_3, will determine the vertical reactions of the arch for a given load. For this reason Polygon p_3 is called the Polygon of the Vertical Reactions.

Polygon p_4.—Similarly, the anti-poles of the various ellipses of elasticity of the voussoirs were located with respect to the horizontal passing through the center of gravity, G_A, of the elastic arch. The statical moments of the elastic weights, with respect to the horizontal through G_A, given in amount by the extensions of the sides of Polygon p_2 cutting the horizontal through G_A, have been applied horizontally to the anti-poles previously mentioned, and, assuming a pole, P_4 (Plate II), the corresponding funicular polygon, p_4, has been drawn. (The Pole P_4 was selected on the vertical, *Y Y*, at a distance from the horizontal through G_A equal to the vertical distance between the parallel first and last sides of Polygon p_3.)

The distance, $\frac{n}{2}$, between the first and last sides of this funicular polygon is proportional to the moment of inertia of the half arch with respect to the horizontal through G_A. After constructing these four polygons, the axis of the ellipse of elasticity of the arch can readily be determined. In fact, the distances of the tangents (vertical and horizontal) to this ellipse are given by:

$$\sqrt{\xi \,.\, v \,.\, n} \text{ and } \sqrt{\lambda_3 \,.\, v}$$

in which,

$\xi = \frac{1}{7} =$ the aliquot part of the pole distance, $2\Sigma_1^{10}\, w$, already mentioned;

$v = 8.32 =$ pole distance, P_4;

$n = 1.46 =$ the part of the horizontal through G_A intercepted between the first and last sides of Polygon p_4; and

$\lambda_3 = 20.00 =$ pole distance of Polygon p_3.

As the arch is symmetrical, the axes of this ellipse are horizontal and vertical, and, therefore, the distances just obtained give the values of the two semi-axes, that is:

$$\rho = \sqrt{\frac{\xi \times \Sigma_1^{20}\, w \times v \times n}{\xi \times \Sigma_1^{20}\, w}} = \sqrt{\xi \times v \times n} = \sqrt{\frac{1}{7} \times 8.32 \times 1.46} = 1.30 \text{ ft.}$$

$$\rho' = \sqrt{\frac{\Sigma_1^{20}\, w \times \lambda_3 \times v}{\Sigma_1^{20}\, w}} = \sqrt{\lambda_3 \times v} = \sqrt{20.00 \times 8.32} = 12.90 \text{ ft.}$$

These axes are not shown on Plate II for lack of room, but appear on Plate IV.

Polygon p_5.—The horizontal statical moments of the elastic weights about the horizontal through G_A, were plotted on the vertical, $Y\,Y$ (Plate II), the pole, or center, P_5, being taken at a distance, n, from the vertical, $Y\,Y$. These statical moments, considered as vertical forces, were applied to the anti-poles of the various ellipses of elasticity with respect to the horizontal through G_A. The funicular polygon, p_5, was then drawn corresponding to Pole P_5.

Polygons p_1, p_3, and p_5 are the characteristics of the arch and, by means of them, its design is readily accomplished. It is seen that these polygons are quite independent of the loading, being determined solely by the geometry of the arch; therefore, they hold good for any hypothesis of loading.

On account of the great importance of these three polygons, it is advisable that the graphical construction be checked by simple computations, according to the following arrangement:

w.	x.	y.	x'.	y'.	x''.	$\frac{w\,x}{\Sigma\,w}$	$\frac{w\,y}{\xi\,\Sigma\,w}$	$\frac{w\,x}{\Sigma\,w} \cdot \frac{x'}{\lambda_3}$	$\frac{w\,y}{\xi\,\Sigma\,w} \cdot \frac{y'}{v}$	$\frac{w\,y}{\xi\,\Sigma\,w} \cdot \frac{x''}{n}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
::	::	::	::	::	::	::::	::::	::::	::::	::::

in which,

$w =$ elastic weight;

$x =$ distance from the center of the voussoir to the axis, Y;

$y =$ distance from the center of the voussoir to the axis, X;

$x' =$ distance between the axis, Y, and the anti-pole of the ellipse of elasticity of the voussoir with respect to the axis, Y;

$y' =$ distance between the axis, X, and the anti-pole of the ellipse of elasticity of the voussoir with respect to the axis, X;

$x'' =$ distance from the axis, Y, to the anti-pole of the voussoir with respect to the axis, X.

It is to be noted that λ_3 is the pole distance for Polygon p_3, and that v and n (Plate II) are, respectively, the summation of Columns (9) and (10). All these distances may be measured to any scale. The elastic weights may be taken as referred to any unit.

Line of Intersection and Envelope Lines.—To recapitulate, Polygon p_1 gives the moment of any force applied to the arch, namely, the product of the force and the segment intercepted on the vertical of the force by the sides of this polygon.

Polygon p_3 gives the right and left reactions at the top of piers, on the basis of the vertical distance between the first and last sides of this polygon as a unit; the amount of vertical force is multiplied by the intercept of this polygon on the vertical of the force applied.

Finally, Polygon p_5 gives the horizontal thrust in the arch due to the same force, which thrust is the product of the force applied and the intercept determined by this polygon on the vertical of the force to the same scale as in

Polygon p_3. In other words, if a vertical force is applied at any point of the arch and its line of action extended through Polygons p_1, p_3, and p_5, its intercepts on these polygons, measured to the proper scale, and multiplied by the force applied, will give respectively, the moment (defined subsequently), the vertical reactions at the supports, and the horizontal thrust. This difference in the scales in measuring the intercepts—that is, moment intercept to the scale of the drawing, left and right reactions, and horizontal thrust to a scale having as a unit the vertical distance between the first and last sides of Polygon p_3—seems at first to be confusing, but in the progress of the design, will prove a substantial simplification of the problem, and avoid a great deal of computation.

In Plate II, ten different positions of the unit load, 1′, 2′, 3′, 4′, 5′, 6′, 7′, 8′, 9′, and 10′, have been assumed. It will be noted that these positions have been selected as having their verticals passing through the center points of the joints of the various voussoirs. This has been done for greater accuracy, because Polygon p_1 and, approximately, Polygons p_3 and p_5, envelop the curves into which these polygons would transform if the length of each voussoir were infinitesimal.

The intersection line, as well as the envelope lines, may be obtained by several methods; the one generally followed in this case is shown in full for Load 5. The intercept between the sides of Polygon p_1, marked $M_{5'}$, is 11.85; therefore, the moment is $M_{5'} = 1 \times \eta_o = R_{A_5'} \times \delta = 1 \times 11.85$.

The left reaction, $R_{A_5'}$ is determined by the construction shown on Plate II, and its value is the quotient of its length divided by the distance between the first and last sides of Polygon p_3, both measured to the scale of the diagram. That is,

$$\delta = \frac{1 \times 11.85}{\dfrac{15.10}{8.32}\ast} = 6.54 \text{ ft.}$$

Therefore, if, with the center at G_A, a circle is described, having this quantity as a radius, and if a tangent to this circle is drawn parallel to the reaction, $R_{A_5'}$, this tangent will represent the position of the left reaction due to Load 5′. In a similar manner, the position of the right reaction, due to the same load, may be determined.

It is clear that the two reactions and the vertical line of action of the load must meet in a point; a series of such points forming a locus or intersection line is shown on Plate II. The reactions left and right, due to Load 5′, tangent to the envelope line, are also shown.

These diagrams (Plate II), are the fundamental part of the arch design, and form a basis for all further computations. Therefore, they should be carefully drawn to as large a scale as possible and checked numerically.

Influence Lines.—Influence lines have been computed for the four sections (numbered I, II, III, and IV) and for loads from Position I to Position II. For each section, the kernel points, m and n, have been determined, and the verticals through these points drawn. Thus, there are two influence lines for

* 8.32 is the length of the vertical intercept between the first and last sides of Polygon p_3 measured to the scale of Plate II.

TABLE 3.—MOMENTS AT KERNEL POINTS FOR ARCH A_2 (WITH FIXED ENDS).

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.247	+ 1.68	+ 2.12	+0.414	+0.523	−0 34	+0.08	−0.083	+0.019	−1.10	−0.72	−0.271	−0.177	−0.88	−0.51	−0.217	−0.125
2	0.867	+ 1.53	+ 1.95	+1.326	+1.690	−0.37	+0.07	−0.320	+0.060	−1.01	−0.62	−0.875	−0.537	−0.69	−0.33	−0.598	−0.286
3	1.631	+ 1.32	+ 1.75	+2.152	+2.854	−0.41	+0.01	−0.668	+0.016	−0.90	−0.55	−1.467	−0.897	−0.40	−0.05	−0.652	−0.081
4	2.253	+ 1.06	+ 1.50	+2 388	+3.379	−0.48	−0.06	−1.081	−0.185	−0.75	−0.40	−1.689	−0.901	+0.02	+0.36	+0.045	+0.811
5	2.425	+ 0.84	+ 1.29	+2.037	+3.128	−0.50	−0.09	−1.212	−0.218	−0.63	−0.25	−1.527	−0.606	+0.32	+0.68	+0.776	+1.649
6	2.507	+ 0.54	+ 1.00	+1.353	+2.507	−0.55	−0.13	−1.378	−0.325	−0.45	−0.08	−1.128	−0.200	+0.79	+1.14	+1.980	+2.857
7	2.425	+ 0.20	+ 0.66	+0.485	+1.600	−0.57	−0.13	−1.382	−0.315	−0.20	+0.20	−0.485	+0.485	......	......	......	......
8	2.253	− 0.24	+ 0.25	−0.540	+0.563	−0.57	−0.10	−1.284	−0.225	+0 25	+0.60	+0.568	+1.851	......	......	......	......
	1.800	......	......	......	......	......	......	......	......	+1.40	......	+2.520	......	......	......	......	......
	1.814	......	......	......	......	......	......	......	......	......	+1.76	......	+3.192	......	......	......	......
9	1.631	− 1.85	− 1.36	−3.017	−2.218	−0.40	+0.10	−0.652	+0.163	+1.26	+1.62	+2.055	+2.642	......	......	......	......
10	0.867	− 4.70	− 4.09	−4.074	−3.546	+1 53	+2.05	+1.326	+1.777	+0.81	+1.20	+0.702	+1.040	......	......	......	......
	0.687	......	......	......	......	+2.90	......	+1.992	......	......	......	......	......	......	......	......	......
	0.706	......	......	......	......	......	+3.30	......	+2.329	......	......	......	......	......	......	......	......
11	0.247	−12.03	−10.96	− 2.971	−2.707	+2.73	+3.11	+0.674	+0.768	+0.60	+0.95	+0.148	+0.234	......	......	......	......

each section, each line being referred to the kernel moment instead of to the center of gravity moment. This method is convenient, for it gives the two moments at each edge of the cross-section directly.

By measuring to scale the intercepts of the verticals through the kernel points between the kernel points and points where the reaction cuts them, and multiplying each intercept by the horizontal thrust (from Polygon p_5), Table 3 has been computed. In Table 3, h_m is the vertical distance from kernel point, m, to the line of the reaction, and M_m is the moment of the reaction about Point m, being the product of h_m and the horizontal component, H, of the reaction. Similarly, h_n and M_n give the vertical distance and moment for the kernel point, n.

It is to be noted that, for Sections II and III, the loads have been placed also on the verticals through the kernel points, m and n; this has been done in order to determine exactly the cuspidal point of each of these influence lines. The results are plotted in Plate III.

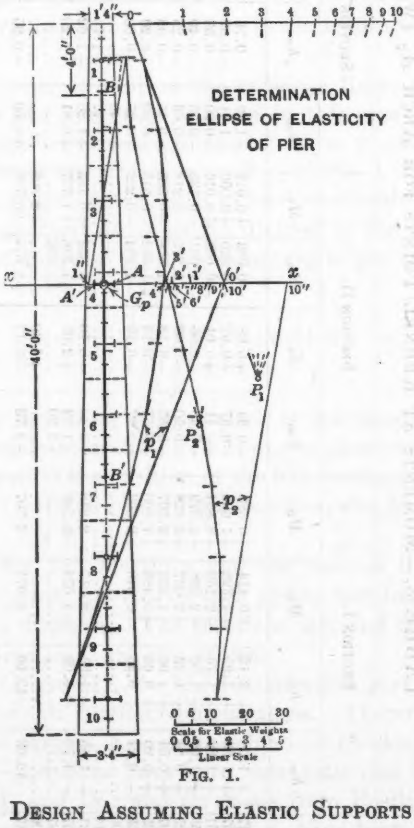
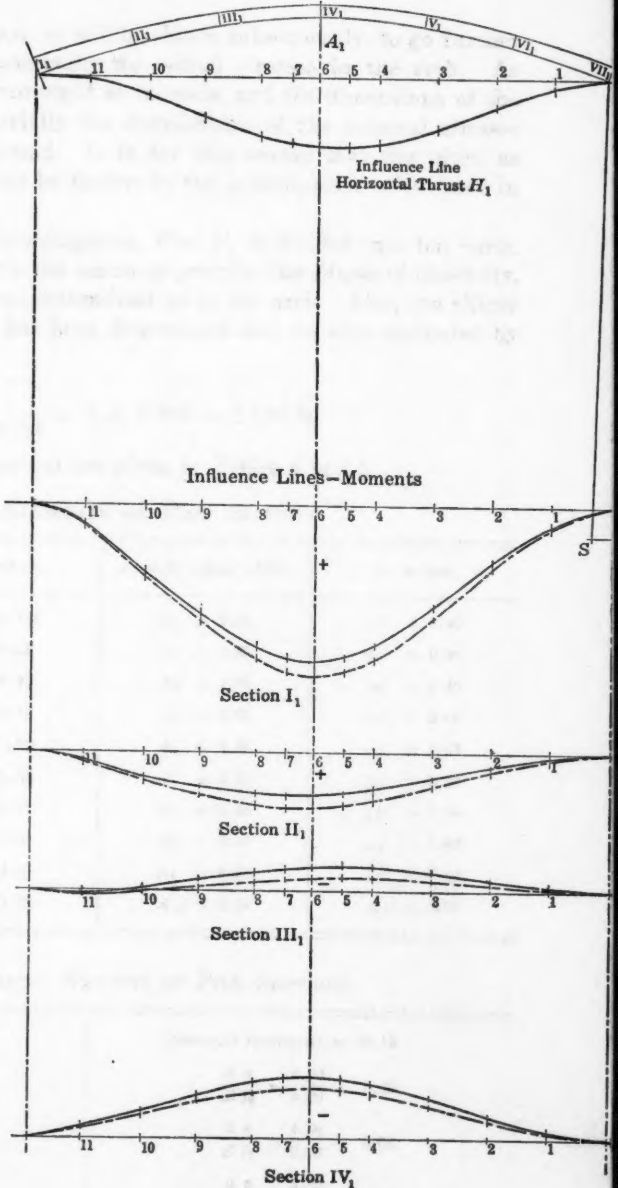


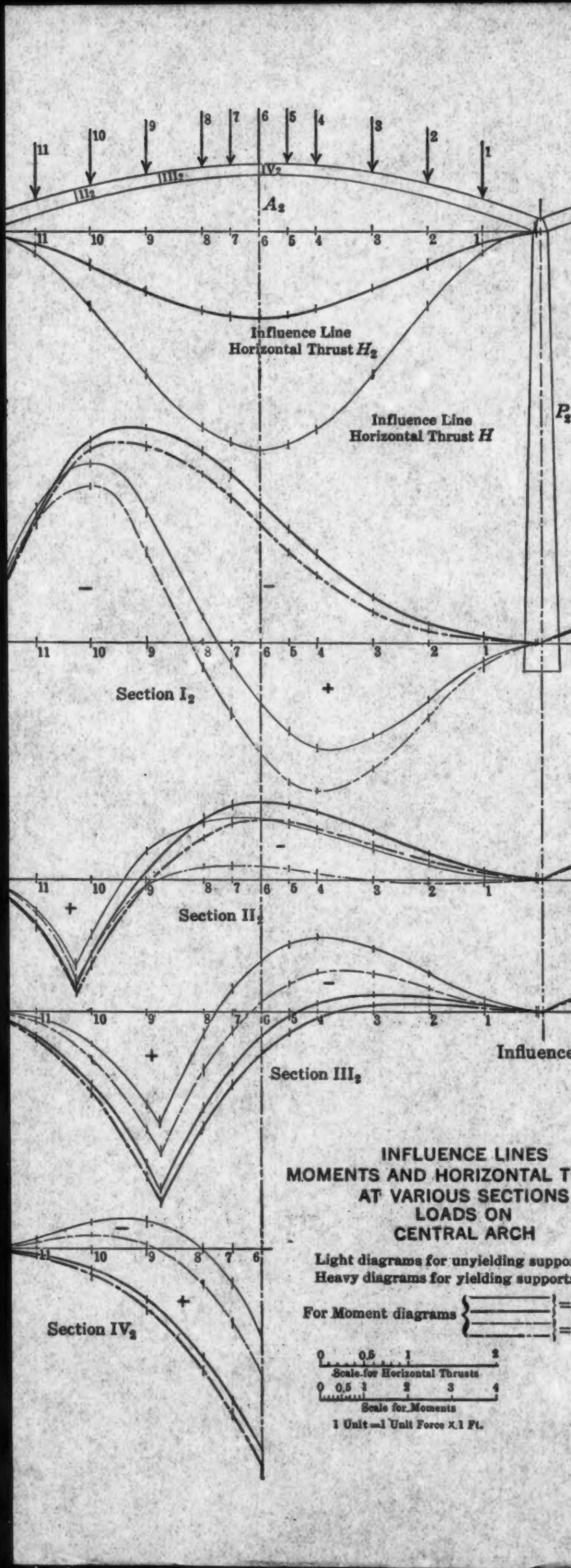
FIG. 1.

DESIGN ASSUMING ELASTIC SUPPORTS

If the arch were to be regarded as having fixed ends, then the design, as far as live load is concerned, would stop here, and the computation of stresses in the sections would be easily made from the influence lines already found. In

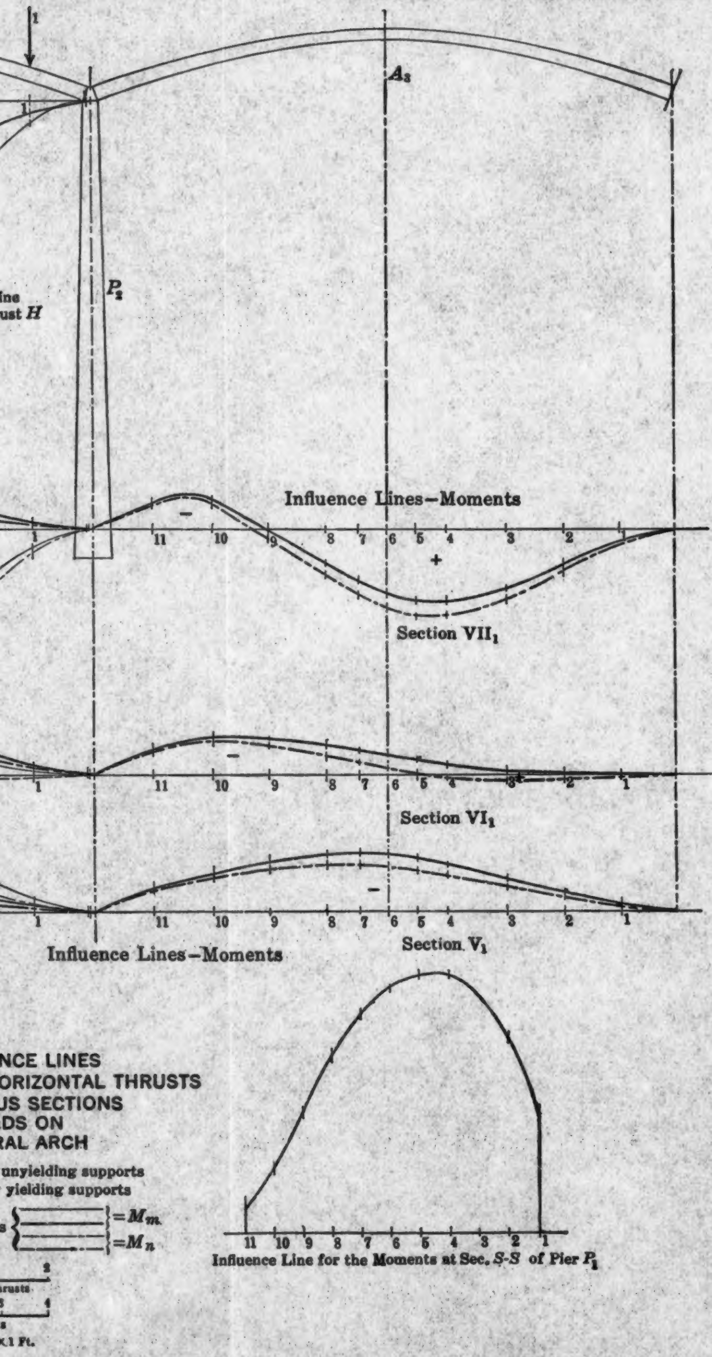


Influence Lines–Moments



11
10
9
8
7
6
5
4
3
2
1
A_2
Influence Line
Horizontal Thrust H_2
Influence Line
Horizontal Thrust H
P_2
Section I_2
Section II_2
Section III_2
Section IV_2
Influence
INFLUENCE LINES
MOMENTS AND HORIZONTAL T
AT VARIOUS SECTIONS
LOADS ON
CENTRAL ARCH
Light diagrams for unyielding suppo
Heavy diagrams for yielding support
For Moment diagrams
Scale for Horizontal Thrusts
Scale for Moments
1 Unit = 1 Unit Force × 1 Ft.

PLATE III.
PAPERS, AM. SOC. C. E.
AUGUST, 1924
JANNI ON
DESIGN OF A MULTIPLE-ARCH SYSTEM



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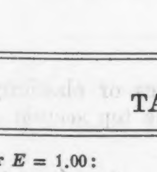
this case, however, there is reason, as will be shown subsequently, to go further in the research, in order to ascertain the actual stresses in the arch. As stated previously, this arch is not rigid at its ends, and the dimensions of the piers are such as to alter materially the distribution of the internal stresses induced in the arch already found. It is for this reason that the piers, as well as the adjacent arches, must be factors in the investigation of stresses in the central arch.

Design of Piers.—For this investigation, Pier P_1 is divided into ten parts, as shown in Fig. 1; for each part, the center of gravity, the ellipse of elasticity, and the elastic weight, have been determined as in the arch. Also, the ellipse of elasticity of the whole pier has been determined and its axis computed by the formula:

$$\rho = 4.0 \times \sqrt{\frac{1}{12}} = 4 \times 0.289 = 1.156 \text{ ft.}$$

The values of the quantities required are given in Tables 4 and 5.

TABLE 4.—Elements of Pier Sections.



TYPICAL CROSS-SECTION OF THE PIER

I, in feet.^{4}	Area, in square feet.	ρ', in feet.
$I_1 = 1.156$	$A_1 = 1.65$	$\rho_1' = 0.40$
$I_2 = 0.33$	$A_2 = 1.75$	$\rho_2' = 0.42$
$I_3 = 0.40$	$A_3 = 1.85$	$\rho_3' = 0.45$
$I_4 = 0.48$	$A_4 = 1.95$	$\rho_4' = 0.49$
$I_5 = 0.58$	$A_5 = 2.05$	$\rho_5' = 0.53$
$I_6 = 0.68$	$A_6 = 2.15$	$\rho_6' = 0.56$
$I_7 = 0.79$	$A_7 = 2.25$	$\rho_7' = 0.59$
$I_8 = 0.91$	$A_8 = 2.35$	$\rho_8' = 0.62$
$I_9 = 1.05$	$A_9 = 2.45$	$\rho_9' = 0.65$
$I_{10} = 1.20$	$A_{10} = 2.55$	$\rho_{10}' = 0.68$

TABLE 5.—Elastic Weights of Pier Sections.

For $E = 1.00$:	(*Brought forward*) = 52.15
$\frac{\Delta s}{E I_1} = \frac{4.00}{0.27} = 14.81$	$\frac{\Delta s}{E I_6} = \frac{4.00}{0.68} = 5.88$
$\frac{\Delta s}{E I_2} = \frac{4.00}{0.33} = 12.12$	$\frac{\Delta s}{E I_7} = \frac{4.00}{0.79} = 5.06$
$\frac{\Delta s}{E I_3} = \frac{4.00}{0.40} = 10.00$	$\frac{\Delta s}{E I_8} = \frac{4.00}{0.91} = 4.39$
$\frac{\Delta s}{E I_4} = \frac{4.00}{0.48} = 8.33$	$\frac{\Delta s}{E I_9} = \frac{4.00}{1.05} = 3.80$
$\frac{\Delta s}{E I_5} = \frac{4.00}{0.58} = 6.89$	$\frac{\Delta s}{E I_{10}} = \frac{4.00}{1.20} = 3.33$
Total	..........74.61

It is to be noted that while for the vertical axis of the ellipse of elasticity of the whole pier it has been possible to apply horizontally the fractional elastic weights, each at its corresponding part of the pier, for the determination of the horizontal axis of the same ellipse, this construction has not been possible, because the centers of the ten sub-divisions lie in the same vertical. In this case, the analytical method has been followed, with the results shown in Table 6.

TABLE 6.—MOMENTS OF INERTIA OF ELASTIC WEIGHTS OF SECTIONS OF PIER ABOUT THE VERTICAL AXIS OF THE PIER.

			(*Brought forward*). =10.457
Point (1)........	$14.8 \times 0.40^2 = 2.368$	Point (6)........	$5.8 \times 0.56^2 = 1.818$
" (2)........	$12.1 \times 0.42^2 = 2.134$	" (7)........	$5.0 \times 0.59^2 = 1.740$
" (3)........	$10.0 \times 0.45^2 = 2.025$	" (8)........	$4.4 \times 0.62^2 = 1.691$
" (4)........	$8.3 \times 0.49^2 = 1.992$	" (9)........	$3.8 \times 0.65^2 = 1.605$
" (5)........	$6.9 \times 0.53^2 = 1.938$	" (10)........	$3.3 \times 0.68^2 = 1.525$
Total..			18.836

Having completed the computations shown in Table 6, and knowing that the pole distance from P_1 to 1-10 is 20 ft., that the pole distances from P_2 to the axis X, is 8 ft., and that the intercept, 0″-10″, on the axis, X, between the first and last sides of Polygon p_2, is 5.54, measured to scale of the elastic weights, the vertical semi-axis of the total ellipse of elasticity of the pier will be given by:

$$\rho = \sqrt{\frac{20 \times 8 \times 55.4}{74.6}} = 10.90 \text{ ft.} = B\,G_P = B'\,G_P$$

and the horizontal semi-axis by:

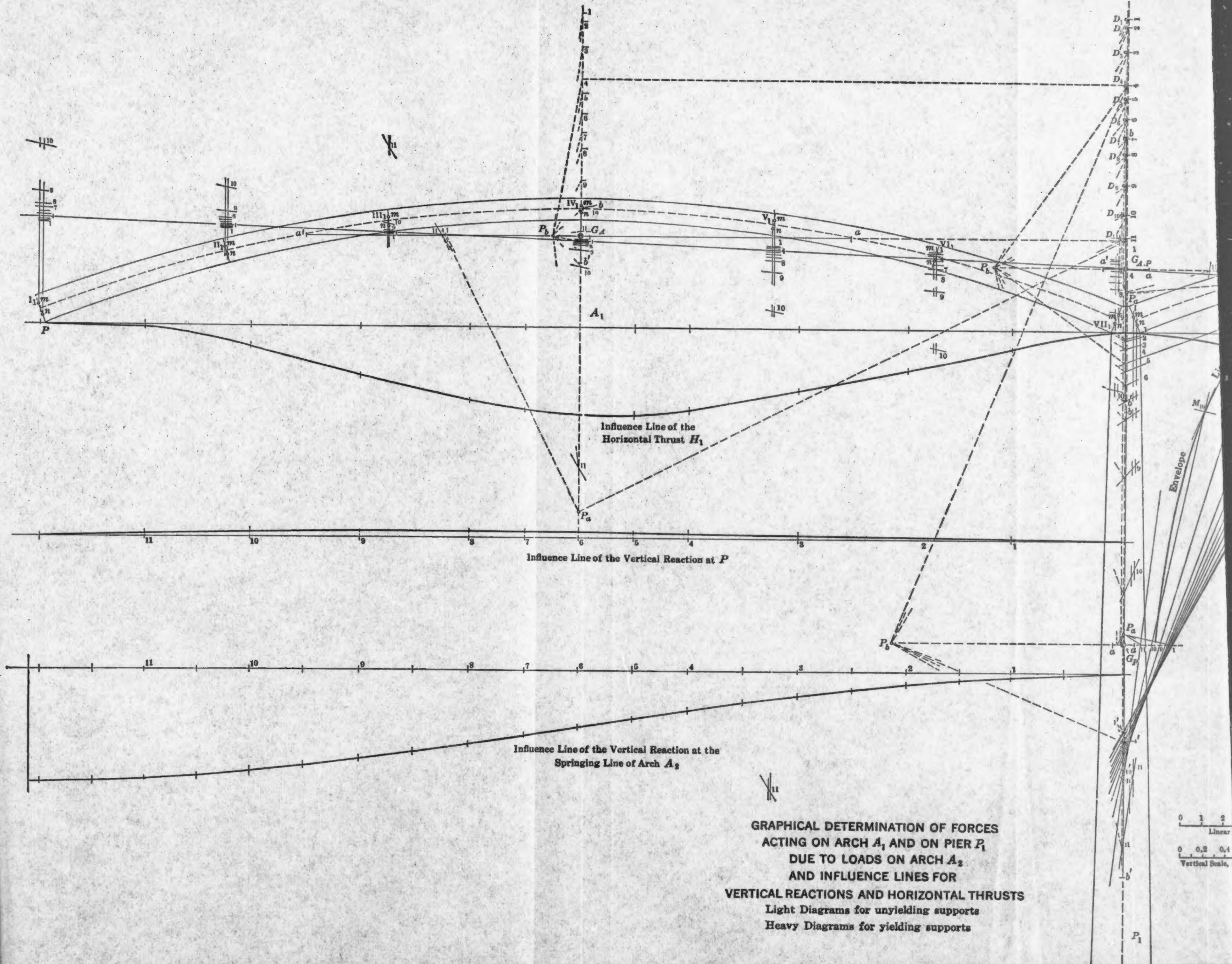
$$\rho' = \sqrt{\frac{18.8}{74.6}} = 0.50 \text{ ft.} = A\,G_P = A'\,G_P$$

The ellipse is then fully determined.

On Plate IV, the respective positions of the two ellipses of elasticity, G_A and G_P, have been drawn to scale, also, their axes, and the top section of the pier.

Combination of Ellipses of Elasticity for Arch and Pier.—Consider the top horizontal section of the pier (Plate IV), and assume that this section is common to the pier, as well as to the arch.* Any horizontal displacement, without rotation, of this section considered as a part of the pier, cannot be induced, according to the theory of the ellipse of elasticity, except by a horizontal force, R_P, applied to the pier and passing through the point, G_P.

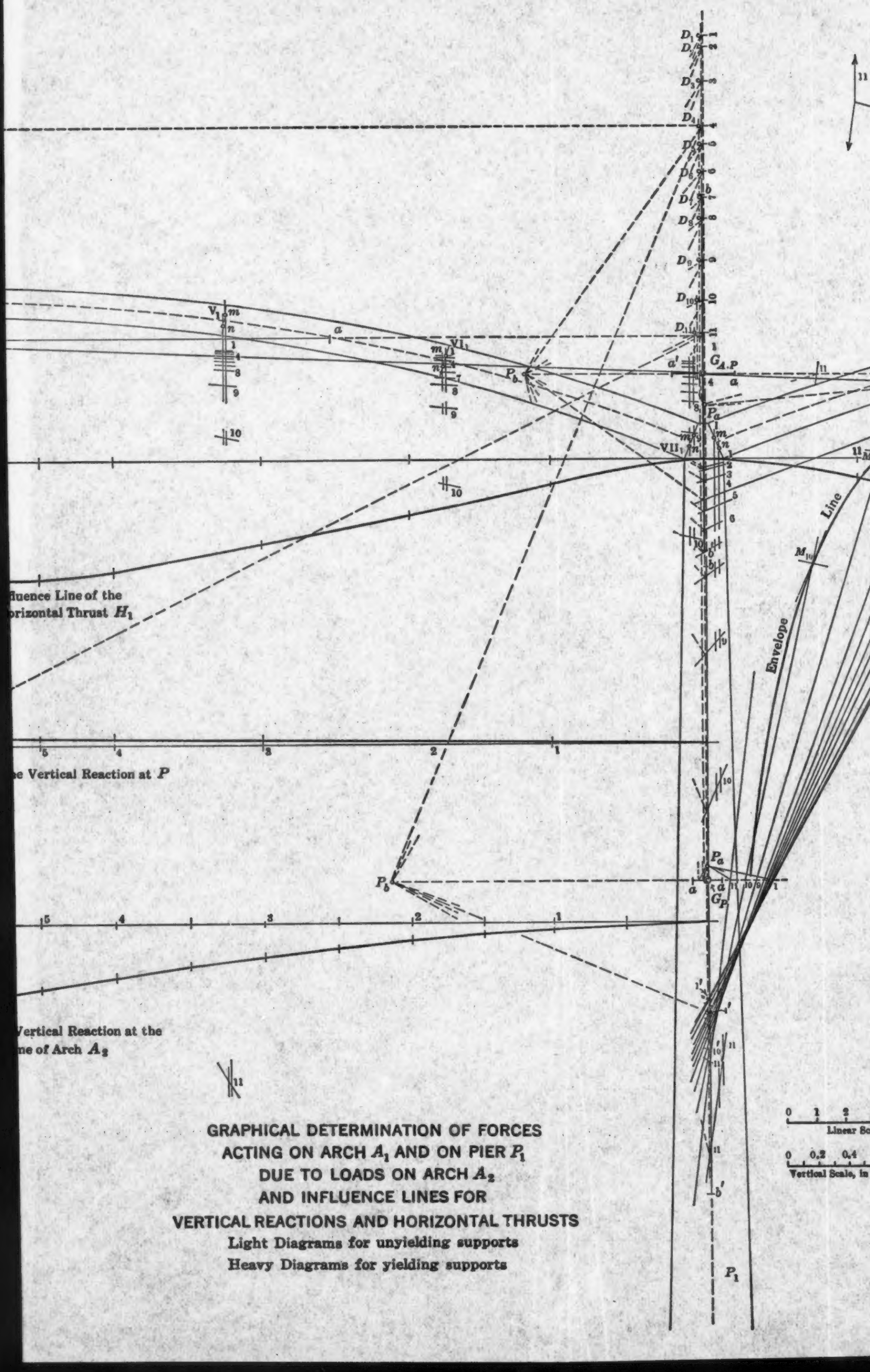
* In considering this section as common to the pier as well as to the arch, the small parallelopiped enclosed within this section and the two end sections of the adjacent arches is disregarded. It is very small and may be ignored without material error. Otherwise, either for the sake of exactness, or because the parallelopiped is of important dimensions with respect to the voussoirs of the arch, it can be taken into consideration very easily by regarding it as an additional voussoir of the actual arch and by determining the relative quantities in the same manner as for the other voussoirs. However, when considering this parallelopiped as a voussoir, actually it is considered twice, once for each arch. In any case, however, the approximation resulting either from disregarding it, or taking it into consideration twice, is quite permissible, for the results, generally speaking, are not affected materially.



GRAPHICAL DETERMINATION OF FORCES ACTING ON ARCH A_1 AND ON PIER P_1 DUE TO LOADS ON ARCH A_2 AND INFLUENCE LINES FOR VERTICAL REACTIONS AND HORIZONTAL THRUSTS

Light Diagrams for unyielding supports

Heavy Diagrams for yielding supports

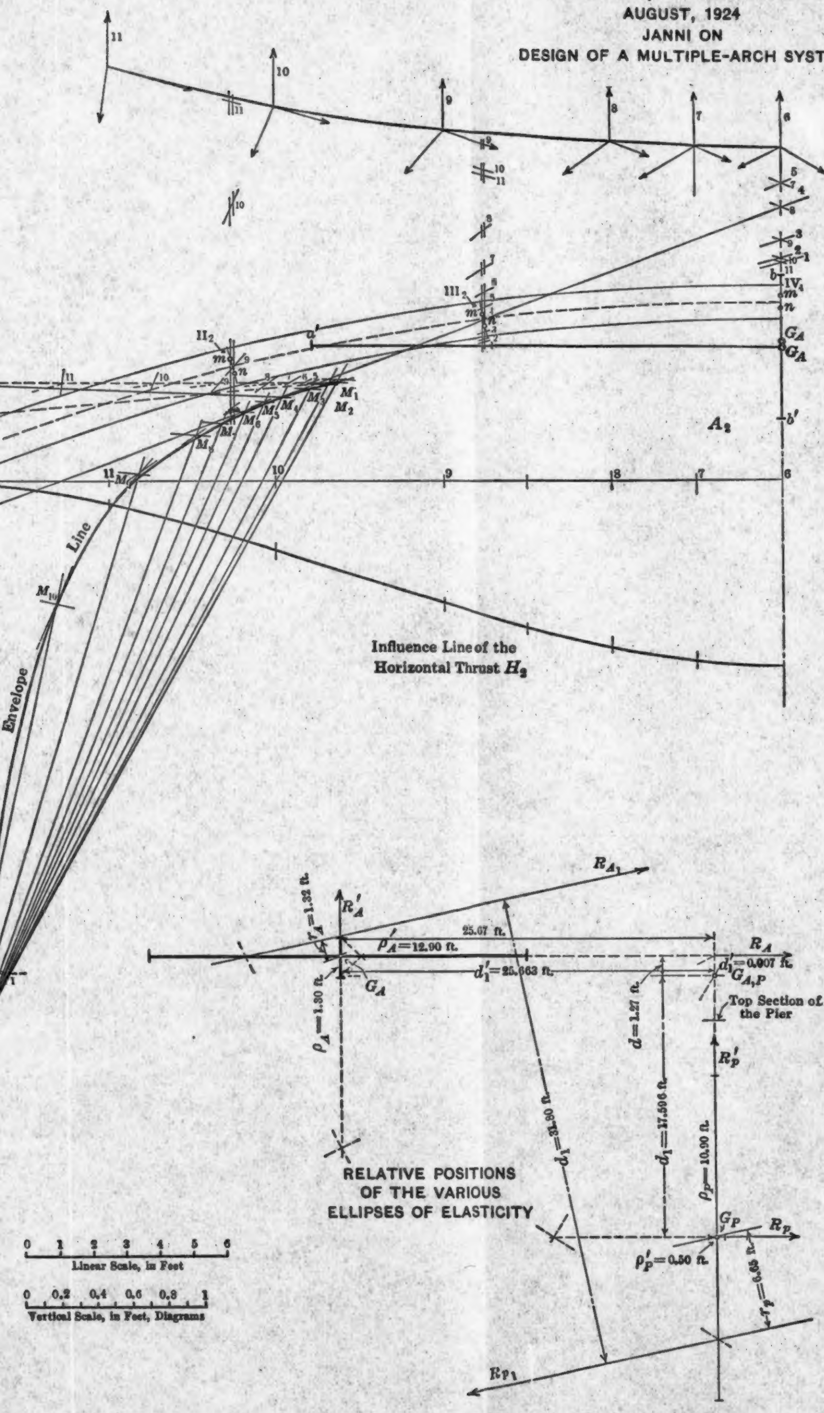


GRAPHICAL DETERMINATION OF FORCES
ACTING ON ARCH A_1 AND ON PIER P_1
DUE TO LOADS ON ARCH A_2
AND INFLUENCE LINES FOR
VERTICAL REACTIONS AND HORIZONTAL THRUSTS

Light Diagrams for unyielding supports

Heavy Diagrams for yielding supports

PLATE IV.
PAPERS, AM. SOC. C. E.
AUGUST, 1924
JANNI ON
DESIGN OF A MULTIPLE-ARCH SYSTEM



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For the same reason, considering this same section as the terminal section of the arch, that same horizontal displacement must be caused by a horizontal force, R_A, applied to the arch and passing through the point, G_A, of the arch.

Considering this same section as an intermediate section of the combined system, G_P, G_A, the same horizontal displacement cannot be caused, except by the resultant of the two forces mentioned previously acting horizontally through a certain point, $G_{A,P}$, which, by the same theory, must be the center of gravity of the combination, G_A, G_P. The displacement of the section, considered as the terminal section of Pier P_1, is given by Equation (1):

$$R_P \times G_P \times \rho_P^2 = R_P \times 74.6 \times 10.90 \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and when considered as a terminal of the arch, it is given by Equation (2):

$$R_A \times G_A \times \rho_A^2 = R_A \times 373.5 \times 1.30 \ldots\ldots\ldots\ldots\ldots (2)$$

In fact, regarding this same section as an intermediate section of the combination, A_1, P_1, the displacement mentioned can be induced on it only by a horizontal force, $R_A + R_P$, passing through the center of gravity, $G_{A,P}$, of the whole system. Therefore, the vertical distances, d_1 and d_2, from the center of gravity, $G_{A,P}$, to the centers, G_A and G_P, respectively, will bear the relation:

$$\frac{d_1}{d_2} = \frac{G_P \times \rho_P^2}{G_A \times \rho_A^2}$$

and, if, $d_1 + d_2 = \delta$:

$$d_1 = \frac{1}{1 + \dfrac{G_A \times \rho_A^2}{G_P \times \rho_P^2}} \times \delta = \frac{1}{1 + \dfrac{373.5 \times 1.30^2}{74.6 \times 10.90^2}} \times 18.86 = 17.596 \text{ ft.}$$

Similarly, with the other co-ordinate of $G_{A,P}$:

$$d_1' = \frac{1}{1 + \dfrac{G_A \times \rho'^2_A}{G_P \times \rho'^2_P}} \times \delta' = \frac{1}{1 + \dfrac{373.5 \times 12.90^2}{74.6 \times 0.50^2}} \times 25.67 = 0.007 \text{ ft.}$$

in which $d'_1 + d'_2 = \delta'$.

The point, $G_{A,P}$, is now fully determined. It is the center of gravity of the entire elastic system, G_A, G_P, as well as the center of the ellipse of elasticity of this combined system. The position of $G_{A,P}$ being determined, the elastic weight of the combined system, $G_{A,P}$, as well as the axes of the ellipse of elasticity of this system,* is found as follows:

A rotation of the same terminal section, considered as an intermediate section of the combined system, A_1, P_1, around the point, $G_{A,P}$, cannot be induced except by a couple. Now, considering this same section as the terminal section of the pier only, the rotation must be induced by a force, R_{P_1}, acting along the anti-polar of $G_{A,P}$, with respect to the total ellipse of elasticity of the Pier P_1; furthermore, the rotation of this same section, considered as a terminal of Arch A_1, must be induced by the force, R_{A_1}, acting along the anti-polar of $G_{A,P}$, with respect to the ellipse of elasticity of the arch. Therefore, the forces, R_{A_1}, and R_{P_1}, constitute a couple; that is, $R_{A_1} = - R_{P_1} = R$, and the amount of rotation is, $\phi = R \times d \times G_{A,P}$. However, considering the arch and the pier

* See also, Prof. Hool's book on this subject.

separately, the same theory of the ellipse of elasticity for the same rotation, ϕ, gives, $\phi = R \times G_A \times r_A = R \times G_P \times r_P$. Therefore:

$$G_{A,P} = \frac{G_A \times r_A}{d} = \frac{G_P \times r_P}{d}$$

that is:

$$G_{A,P} = \frac{373.54 \times 1.32}{31.80} = 15.50$$

$$G_{A,P} = \frac{74.61 \times 6.65}{31.80} = 15.60^*$$

Assume that $G_{A,P} = 15.55$ as an average, which is the value of the elastic weight of the combined system, A_1, P_1.

Now, considering the horizontal displacement of the top section of Pier P_1 when this is regarded as an intermediate section of the combined system, A_1, P_1, the value of its horizontal displacement may be expressed by $(R_A + R_P)\, G_{A,P} \times \rho^2$. From Equation (1):

$$\rho^2 = \frac{G_P \times \rho_P^2}{G_{A,P}\left(1 + \dfrac{G_P \times \rho_P^2}{G_A \times \rho_A^2}\right)} = \frac{G_P \times \rho_P^2}{G_{A,P}\left(1 + \dfrac{d_1}{d_2}\right)} = \frac{74.61 \times 10.9^2}{15.55\left(1 + \dfrac{17.59}{1.26}\right)}$$

$$= 38.10;\ \rho = 6.17 \text{ ft.}$$

This equation will give the distances from the horizontal tangents of the ellipse of elasticity of the combined system, A_1, P_1, to Point $G_{A,P}$. In an identical manner, the position of the vertical tangents to the same ellipse are determined by:

$$\rho'^2 = \frac{G_P \times \rho'^2_P}{G_{A,P}\left(1 + \dfrac{d'_1}{d'_2}\right)} = \frac{74.61 \times 0.50^2}{15.55\left(1 + \dfrac{0.007}{25.66}\right)} = 1.19;\ \rho' = 1.09 \text{ ft.}$$

Finally, as the displacement of the same section considered previously when it is regarded as an intermediate section of the system, A_1, P_1, must be perpendicular to a direction that is conjugate (with respect to the ellipse, $G_{A,P}$) to the line of action of the force inducing that displacement, so for this ellipse, the vertical and horizontal directions are conjugate to each other, and, therefore, the points at which these tangents will cut the axes of the ellipse will be also the ends of these axes.

Design of Arch.—There now exists a new elastic system composed of the former elastic arch with two elastic systems supporting it, one at either end, each comprising a pier and an adjacent arch, and each represented by the ellipse of elasticity just determined. It is clear, therefore, that if in designing the arch, the ellipses of elasticity of the supports are assumed as if they were the ellipses of elasticity of two ideal additional voussoirs—in other words, considering the arch as composed of twenty-two voussoirs instead of twenty, as has been done previously—the resulting stresses will be for an arch under the influence of the elasticity of its supports (Arch A_1, Pier P_1 on the left; Arch A_3, Pier P_2 on the right).

* Theoretically speaking, these two values should have been the same, but unavoidable inaccuracy of drafting gives this small and unimportant difference.

In Plate II is shown in heavy lines (for distinction) the design of this new elastic system under the assumption that the voussoirs of the half arch number eleven instead of ten as before. The method followed in determining these diagrams is identical with that for the case that assumed unyielding supports.

Influence Lines for Moments.—In Plate III, the various diagrams (light and heavy) for the center span have been plotted. Inspection of these diagrams will reveal the great difference in the moments acting on the central arch of the system under the two assumptions.

Eleven positions have been considered for the unit load, which is supposed to move from right to left as indicated numerically. Knowing (1) the horizontal thrust for each of these positions; (2) the vertical distances between the kernel points of the various sections, and the points on the verticals passing through these kernels, in which the reactions cut these verticals, the moments acting on the various sections have been computed as shown in Table 7.

Design of Side Arch and Pier

Thus far, the influence lines of the moments have been considered only for the central arch. In order to complete this research, the influence lines for the side arches, as well as for the piers, were determined on the assumption that only the central arch was loaded.

Reactions for Side Arch and Pier.—From Plate II were obtained the reactions of the elastic supports of the arch for each loading of the central arch. These results are plotted on Plate III, showing the stress acting on Arch A_1* and Pier P_1, while the central arch is being loaded. Each of these reactions evidently represents the combined action of the elastic system, G_A, G_P, supporting the arch and reacting against the middle arch under loading. In order to determine which part of this reaction belongs to the arch and which to the pier, the following construction was adopted:

A rotation, without parallel displacement of the top section of the pier considered as an intermediate section of the elastic system, G_A, G_P, when this system is under the action of a force, was performed about a center of instantaneous rotation, D. This point, according to the theory of the ellipse of elasticity is the anti-pole of the line of the acting force with respect to the ellipse of elasticity of the whole system, $G_{A,\,P}$. The rotation of this section considered as one end of Pier P_1, can be induced only by a certain force, the line of action of which is the anti-polar of the center, D, this time with respect to the ellipse of elasticity, G_P, of the pier. This determines the line of action of the component of the reaction due to the pier under a certain loading. Furthermore, the rotation of the same section, considered as a terminal of the arch, is induced by a force the line of action of which is the anti-polar of the same center, D, this time with respect to the ellipse of elasticity, G_A, of the arch. The line of action, therefore, of the component of the reaction due to the arch, is likewise determined. As a check on this construction, these two lines, together with the reaction, must meet in a point. Knowing the

* The diagrams for Arch A_1 hold good for the similar sections of Arch A_2.

TABLE 7.—MOMENTS AT KERNEL POINTS FOR ARCH A_2 (WITH ELASTIC SUPPORTS).

Position of load.	H_2.	Section I_2.				Section II_2.				Section III_2.				Section IV_2.			
		h_m.	h_n	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.132	− 1.06	− 0.60	−0.139	−0.079	−1.49	−1.09	−0.196	−0.143	−0.86	−0.50	−0.113	−0.066	+1.03	+1.36	+0.135	+0.179
2	0.381	− 1.14	− 0.69	−0.434	−0.262	−1.51	−1.11	−0.575	−0.422	−0.81	−0.44	−0.308	−0.167	+1.16	+1.49	+0.441	+0.567
3	0.677	− 1.52	− 1.04	−1.029	−0.704	−1.54	−1.14	−1.042	−0.771	−0.60	−0.23	−0.406	−0.155	+1.69	+2.02	+1.144	+1.367
4	0.913	− 2.18	− 1.69	−1.990	−1.542	−1.67	−1.26	−1.524	−1.150	−0.19	+0.16	−0.173	+0.146	+2.67	+3.00	+2.437	+2.739
5	0.983	− 2.56	− 2.07	−2.516	−2.034	−1.69	−1.28	−1.661	−1.258	+0.14	+0.51	+0.137	+0.501	+3.40	+3.78	+3.342	+3.666
6	1.009	− 3.15	− 2.67	−3.178	−2.694	−1.75	−1.31	−1.765	−1.321	+0.61	+0.99	+0.615	+0.998	+4.49	+4.82	+4.530	+4.863
7	0.983	− 3.90	− 3.37	−3.833	−3.312	−1.71	−1.28	−1.680	−1.258	+1.31	+1.69	+1.287	+1.661	......	......	......	......
8	0.913	− 4.71	− 4.18	−4.300	−3.816	−1.49	−1.04	−1.360	−0.949	+2.44	+2.83	+2.227	+2.583	......	......	......	......
	0.749	......	......	......	......	......	......	......	......	+5.42	......	+4.059	......	......	......	......	......
	0.751	......	......	......	......	......	......	......	......	......	+5.75	......	+4.318	......	......	......	......
9	0.677	− 7.19	− 6.60	−4.867	−4.468	−0.30	+0.17	−0.203	+0.115	+5.10	+5.44	+3.452	+3.682	......	......	......	......
10	0.381	−12.27	−11.50	−4.674	−4.381	+4.58	+5.16	+1.744	+1.965	+4.33	+4.67	+1.649	+1.779	......	......	......	......
	0.308	......	......	......	......	+7.86	......	+2.420	......	......	......	......	......	......	......	......	......
	0.315	......	......	......	......	......	+8.20	......	+2.588	......	......	......	......	......	......	......	......
11	0.132	−21.20	−20.77	−2.798	−2.741	+7.73	+8.09	+1.020	+1.067	+4.16	+4.50	+0.549	+0.594	......	......	......	......

intensity of the reaction and the lines of action of the components just determined, their amount is found. Changing the signs of these two components gives the forces acting on the side arch and on the pier.

In Plate IV is shown the construction required for these determinations as regards Position (4) of the load, while for other positions merely the results are indicated.

The procedure for Position (4) of the load will be considered in detail. From Plate II, the position of the left reaction, due to Load 4, as well as the envelope line and the intersection line, have been drawn to scale on Plate IV. This reaction has been extended to meet the axes of the ellipse, $G_{A,P}$, as shown, and the anti-pole, D_4, with respect to the ellipse, $G_{A,\,P}$ has been determined.

Considering D_4 as an anti-pole with respect to the ellipse, G_P, of the pier, the polar of this anti-pole may be determined—in this case, M_4-4′. Finally, considering this same D_4 as an anti-pole with respect to the ellipse, G_A, its corresponding polar may be drawn as shown in full in Plate IV; as was to be expected, it passes through Point M_4, which checks the geometrical construction. The two lines so determined will represent the reactions due to the pier and to the arch, respectively.

As in the previous case, having determined the amount and direction of the reaction on the arch, A_1, its horizontal and vertical components may be obtained. The horizontal component will represent the horizontal thrust of the arch, and the vertical component will represent the vertical reaction of the left support of Arch A_1. This operation being performed for every force acting on the arch gave the horizontal thrust acting on the arch, as well as the left reaction of Arch A_1, for each position of the unit load acting on Arch A_2. These results are shown on Plate IV.

The horizontal thrust on Arch A_1 being known for each position of the load on Arch A_2, the relative moments acting on the sections of Arch A_1 were computed, by multiplying, in each instance, the horizontal thrust by the distance between the kernel points of the sections under consideration and the line of action of the force; these distances were measured on the verticals passing through the kernel points of the section under investigation. These computations were made for Sections I_1 to VII_1, inclusive (see Table 8), and the corresponding influence lines of the moments were plotted on Plate III.

The vertical reaction of the support, P_1, has also been plotted; this is merely Polygon p_3 of Plate II developed and drawn to scale. By repeating for Pier P_1 the operations already described for Arch A_1, the horizontal components of the forces were computed and the moments acting on the pier found. These moments are shown on Plate III. Inspection of Plate III, shows at a glance, that there is a remarkable difference in the results obtained by the two different hypotheses.

Yielding of Soil.—It should be noted that in this problem, the yielding of the soil has not been considered, because of the light form of construction contemplated, and of the possibility of stabilizing the foundation even if the soil were somewhat yielding.

However, in case of peculiar conditions such as a somewhat yielding soil and no means of making it rigid, the elasticity of the foundation should be

TABLE 8.—MOMENTS AT KERNEL POINTS FOR ARCH A_1 UNDER LOADS ON ARCH A_2.

Position of load.	H_1.	Section I_1.				Section II_1.				Section III_1.				Section IV_1.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.115	+ 3.94	+ 4.36	+0.453	+0.501	+0.91	+1.27	+0.104	+0.146	−0.79	−0.44	−0.090	−0.050	− 1.60	− 1.27	−0.184	−0.146
2	0.322	+ 3.97	+ 4.38	+1.278	+1.410	+0.94	+1.29	+0.302	+0.415	−0.77	−0.42	−0.247	−0.135	− 1.62	− 1.29	−0.521	−0.415
3	0.555	+ 4.03	+ 4.45	+2.236	+2.469	+0.97	+1.33	+0.538	+0.738	−0.75	−0.40	−0.416	−0.222	− 1.64	− 1.31	−0.910	−0.727
4	0.735	+ 4.19	+ 4.59	+3.079	+3.373	+1.05	+1.42	+0.771	+1.043	−0.73	−0.38	−0.536	−0.279	− 1.67	− 1.34	−1.227	−0.984
5	0.775	+ 4.27	+ 4.69	+3.309	+3.634	+1.11	+1.48	+0.860	+1.147	−0.69	−0.34	−0.534	−0.263	− 1.69	− 1.36	−1.309	−1.054
6	0.770	+ 4.42	+ 4.83	+3.403	+3.719	+1.19	+1.56	+0.916	+1.201	−0.67	−0.31	−0.515	−0.238	− 1.75	− 1.42	−1.347	−1.093
7	0.730	+ 4.59	+ 5.00	+3.350	+3.650	+1.30	+1.67	+0.949	+1.219	−0.65	−0.29	−0.474	−0.211	− 1.79	− 1.46	−1.306	−1.065
8	0.652	+ 4.77	+ 5.16	+3.110	+3.364	+1.38	+1.74	+0.899	+1.134	−0.62	−0.27	−0.404	−0.176	− 1.86	− 1.53	−1.212	−0.997
9	0.435	+ 5.36	+ 5.77	+2.331	+2.509	+1.69	+2.04	+0.735	+0.887	−0.58	−0.22	−0.252	−0.095	− 2.11	− 1.78	−0.917	−0.774
10	0.185	+ 7.51	+ 7.90	+1.389	+1.461	+2.84	+3.18	+0.525	+0.588	−0.30	+0.04	−0.055	+0.007	− 2.88	− 2.55	−0.532	−0.471
11	0.020	+17.84*	+17.94†	+0.356	+0.358	+9.00*	+9.11†	+0.180	+0.182	+3.35	+3.62	+0.067	+0.072	−12.23	−11.90	−0.244	−0.238

TABLE 8.—(*Continued*).

Position of load.	H_1.	Section V_1.				Section VI_1.				Section VII_1.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.115	− 1.26	− 0.91	−0.144	−0.104	0	+ 0.36	0	+0.041	+ 2.52	+ 2.94	+0.289	+0.338
2	0.322	− 1.28	− 0.93	−0.412	−0.299	− 0.03	+ 0.33	−0.009	+0.106	+ 2.48	+ 2.90	+0.798	+0.933
3	0.555	− 1.33	− 0.98	−0.738	−0.543	− 0.10	+ 0.26	−0.055	+0.144	+ 2.37	+ 2.79	+1.315	+1.548
4	0.735	− 1.44	− 1.08	−1.058	−0.793	− 0.28	+ 0.09	−0.205	+0.066	+ 2.14	+ 2.56	+1.572	+1.881
5	0.775	− 1.50	− 1.14	−1.162	−0.883	− 0.37	− 0.01	−0.286	−0.007	+ 2.02	+ 2.44	+1.565	+1.891
6	0.770	− 1.63	− 1.27	−1.255	−0.977	− 0.55	− 0.18	−0.423	−0.138	+ 1.77	+ 2.19	+1.362	+1.686
7	0.730	− 1.76	− 1.41	−1.284	−1.029	− 0.75	− 0.38	−0.547	−0.277	+ 1.49	+ 1.92	+1.087	+1.401
8	0.652	− 1.93	− 1.57	−1.258	−1.023	− 0.99	− 0.62	−0.645	−0.404	+ 1.15	+ 1.58	+0.749	+1.030
9	0.435	− 2.48	− 2.12	−1.078	−0.922	− 1.80	− 1.44	−0.783	−0.626	+ 0.06	+ 0.50	+0.026	+0.217
10	0.185	− 4.27	− 3.91	−0.789	−0.723	− 4.47	− 4.08	−0.826	−0.754	− 3.58	− 3.14	−0.662	−0.580
11	0.020	−26.65	−26.19	−0.533	−0.523	−19.54*	−19.29†	−0.390	−0.385	−25.48*	−25.12†	−0.509	−0.502

* Normal distance between m and line of action of "11".
† Normal distance between n and line of action of "11".

considered in designing an arch. This does not complicate the problem in any way; it only means the addition of an ellipse, due to the yielding of the soil,* to the various ellipses of elasticity of the pier sections. The designer should investigate the behavior of a multiple-arch system in every case where the importance of the spans, the comparative slenderness of the piers, and the conditions of the foundation are such as to warrant a thorough investigation.

Dead Loads.—The polygon of pressure due to dead load, which, in this case, is the same for all three arches, as all are equal and equally loaded, is not affected by the two hypotheses under which Arch A_2 and, consequently, Arches A_1 and A_3 were designed. Therefore, it will not be discussed here.

Temperature.—The effect of temperature change, as well as that of shrinkage in concrete due both to the setting of the concrete and to the dead and live loads, is not presented in this paper. The problem has been solved only for the live load, and in a manner, which, in the present state of knowledge, represents the most complete solution that can be desired.

Conclusions

From the results obtained in this investigation, as represented by the diagrams of Plate III, the following conclusions seem to be justified.

The conditions assume an elastic system composed of several arches, supported by comparatively slender piers, each arch being regarded as having fixed ends. According to theory, this elastic system should be considered as a unit by itself, and the design carried out accordingly. The analytical methods that should be followed, in this instance, would be exceedingly cumbersome and laborious, and the graphical method, which has been applied in the case of three arches, although very simple and elegant, would become rather laborious in application if extended to several arches.

The question arises, therefore, whether it is possible, for practical purposes, to simplify or shorten this task of the designer in a case like that mentioned. Professor Guidi, of the Polytechnic of Turin, Italy, has suggested a simplification which would eliminate a great deal of work for the designer. He would consider three adjacent arches of the system as forming an elastic system by itself, disregarding what is on either side of this selected unit, that is, the outer ends of this three-arch system should be regarded as fixed. This assumption, however, would only be strictly correct if all the members of the system were of the same dimensions.

Considering the influence lines of Plate III, the statement is justified that if the arches of this system, no matter how many, are reinforced in such a way as to withstand the conditions of loading, as shown in this paper, the system is to be regarded as safely designed as far as live load is concerned; and if the piers are reinforced to withstand the condition of loading, as shown for Arch A_2, they are safe.

* As explained in Prof. Hool's book, previously mentioned.

APPENDIX

Graphical Treatment of Statical Moments, Products of Inertia, Moment of Inertia, and Ellipse of Inertia, Leading to the Conception and Construction of the Ellipse of Elasticity. Method to Be Followed in Applying this Theory to a Plane Elastic System*

General Principles

1.—Statical Moment.—Let F_1, F_2, and F_3 (Fig. 2) be three parallel coplanar forces. To obtain the statical moments of these three forces with respect to a given axis, x, construct the equilibrium polygon on F_1, F_2, and F_3, with respect to the polygon of forces, 0, 1, 2, and 3, and the pole, P. The segments, $0'1'$, $1'2'$, and $2'3'$, are proportional to the "statical moments" desired.

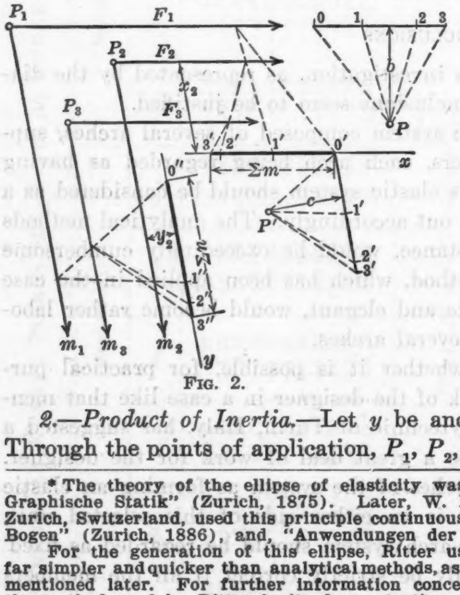

FIG. 2.

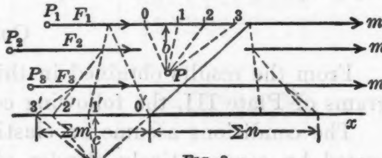

FIG. 3

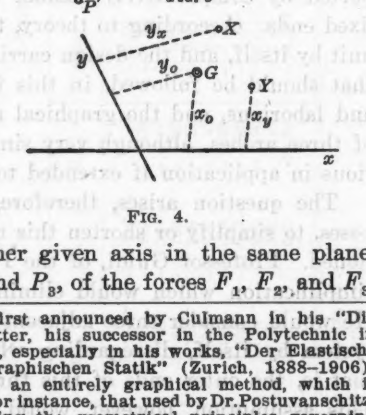

FIG. 4.

2.—Product of Inertia.—Let y be another given axis in the same plane. Through the points of application, P_1, P_2, and P_3, of the forces F_1, F_2, and F_3,

* The theory of the ellipse of elasticity was first announced by Culmann in his "Die Graphische Statik" (Zurich, 1875). Later, W. Ritter, his successor in the Polytechnic in Zurich, Switzerland, used this principle continuously, especially in his works, "Der Elastische Bogen" (Zurich, 1886), and, "Anwendungen der Graphischen Statik" (Zurich, 1888–1906).

For the generation of this ellipse, Ritter uses an entirely graphical method, which is far simpler and quicker than analytical methods, as, for instance, that used by Dr. Postuvanschitz, mentioned later. For further information concerning the geometrical principles governing the method used by Ritter in its demonstration, see "Elements of Projective Geometry," by L. Cremona, translated by Charles Leudesdorf, 1885, Clarendon Press, Oxford.

Since then, this theory has been extensively taught and used in Europe, especially for the solution of the most difficult problems of engineering, where the analytical methods are found impractical.

Besides the books mentioned, there are others worthy of note, dealing with this subject, among which, for instance, may be mentioned: "Teoria dell'Elasticita' e Resistenza dei Materiali," Parte Seconda, and "L'arco Elastico senza Cerniere," by Camillo Guidi, 1903; "Contributo alla Trattazione Grafica dell'arco Continuo su Appoggi Elastici," by M. Panetti, 1901; and "Fortschritte der Engenieurwissenschaften," by Dr. F. Postuvanschitz; the author of this book deals with this subject mainly by analysis; Prof. H. Lossier, in 1913, also published in *Genie Civil* a paper dealing with the application of this method to a system of arches.

In May, 1913, the writer presented a paper before the Western Society of Engineers, *Proceedings*, Vol. XVIII, No. 5, dealing with this subject, under the title, "Designing of an Arch System," which work seems to have been the first of its kind written in the English language. In the third volume of "Reinforced Concrete Construction," by Prof. G. A. Hool, there is a treatment of this graphical method. In April, 1917, S. Moreell, Jr., also presented a paper before the Western Society of Engineers dealing on this subject. A learned criticism of this theory is given by George F. Swain, Past-President, Am. Soc. C. E., in a paper entitled "On a New Principle in the Theory of Structures," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 702 *et seq.*

draw lines parallel to y. Assume forces of the same magnitude as $0'1'$, $1'2'$, and $2'3'$, the statical moments with respect to the axis, x, to lie, respectively, in these lines parallel to y, and construct on them the equilibrium polygon the pole of which is P' (Fig. 2), giving the segments, $0''1''$, $1''2''$ and $2''3''$, on the line, y, which are proportional to the products of inertia of the forces, F_1, F_2, and F_3, with respect to the axes, x and y. The following equation gives for any one of the forces, say, F_2, the double moment called the "product of inertia", with respect to the axes, x and y:

$$\text{Product of Inertia of } F_2 = F_2 . x_2 . y_2 = b . c . 1'' 2''$$

in which b and c are the polar distances. Also, for the whole system, $\Sigma F x y = b . c . \Sigma n$ (Fig. 2).

3.—Moment of Inertia.—If the axis, y, coincides with the axis, x, the product of inertia assumes the special value called the "moment of inertia".

Let F_1, F_2, and F_3 be three parallel co-planar forces. To obtain the moment of inertia with respect to a given axis, x, a procedure analogous to that of Paragraph 2 is followed (Fig. 3). The moment of inertia of the whole system $= \Sigma F x^2 = b . c . \Sigma n$.

The expression, $\Sigma F x^2 : \Sigma F$, is of the second degree; hence, it can be represented by the square of a length, r; this is what is called the "radius of inertia" (Radius of Gyration), therefore:

$$r^2 = \frac{\Sigma F x^2}{\Sigma F}$$

Let x be an axis lying in the plane, and x_0, an axis parallel to x, passing through the center of gravity of the system; d, the distance between these two axes; and x and x_0, the distances from P, a point of application of one of the forces of the system to these axes (these distances, d, x, and x_0 are taken in an arbitrary direction); $x = x_0 \pm d$, and, therefore, knowing that $\Sigma F x_0 = 0$,

$$\Sigma F x^2 = \Sigma F x_0^2 + d^2 . \Sigma F \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

that is:

The moment of inertia of a system with respect to any axis is equal to the moment of inertia of this system with respect to its gravity axis parallel to the given axis plus the product of the summation of the forces and the square of the distance between the two axes.

If r and ρ are the radii of inertia of the system with respect to Axes x and x_0, then $r^2 = \rho^2 + d^2$, that is:

The square of the radius of inertia with respect to any axis is equal to the square of the radius of inertia with respect to the gravity axis parallel to the given axis plus the square of the distance between the two axes.

4.—Center of Gravity of Static Moments.—Any system of forces, such as F_1, F_2, and F_3, has a center of gravity; so also, considering their statical moments about any axis in their plane as a new system of forces, the latter must have a center of gravity. This is called the center of gravity of the statical moments, or briefly, "the center relative to that axis". Its position depends on that of the axis of moments. To locate this center relative to the axis, lay off lines representing to scale the magnitudes of the static moments considered as forces, and proceed precisely as in finding the center of gravity of a plane system of forces. Subsequently, several important properties of such centers are given.

5.—Conjugate Axes.—In Fig. 4, let G be the center of gravity of a system of forces, X, the center relative to the axis, x, and Y, the center relative to the axis, y (that is, X is the center of gravity of the statical moments of the forces about the axis, x; Y, that about y).

Also, let the distances from G, X, and Y, to the axes be computed in any assumed direction; then, since the sum of the statical moments of the forces of the system with respect to the axis, x, equals the moment of the resultant with respect to the same axis:

$$\Sigma F x = x_0 . \Sigma F$$

Then, as explained previously, if the individual products, $F . x$, are regarded as forces, and their static moments with respect to y found, the following equation may be written:

$$\Sigma (F x) y = \Sigma F x y = y_x . \Sigma F x$$

Substituting, further, the value for $\Sigma F x$:

$$\Sigma F x y = y_x . x_0 . \Sigma F$$

If the order of computing the statical moments is reversed:

$$\Sigma F y x = x_y . y_0 . \Sigma F$$

Combining the last two equations:

$$y_x . x_0 = x_y . y_0$$

If neither x nor y passes through G, that is, neither x_0 nor y_0 is zero, then, to satisfy this equation, if y_x is zero, x_y will also be equal to zero. Hence:

If the center relative to an axis falls on another axis, the center relative to the latter will fall on the former.

Again, if an axis, y, passes through a center, X, its relative center, Y, will fall on the axis, x, of which X is the relative center. Therefore, any axis passing through the given point, X, the relative center of the axis, x, will have its relative center lying on x. Hence, to each line of the set of lines in the plane considered as passing through X (pencil of rays in the language of projective geometry), will correspond a definite point of a set of points (range) on the axis, x, of which X is the relative center. There is, therefore, a "projective correspondence" between axis and center. Two axes, situated so that each passes through the center (pole) of the other, are called "conjugate axes".

In view of the preceding, either of the equations:

$$\Sigma F x y = y_x . x_0 . \Sigma F$$

or:

$$\Sigma F y x = x_y . y_0 . \Sigma F$$

may be thus stated:

The product of inertia of a plane system of forces with respect to two conjugate axes is zero, and conversely.

6.—Applying this reasoning to the special case of the product of inertia that is called the moment of inertia, the latter may be found with respect to an axis, x, and, calling x_x the distance from the center, X, to the axis, x, Equation (2) is derived:

$$\Sigma F x^2 = x_x . x_0 . \Sigma F \quad \text{.......................(2)}$$

or,

$$x_x = \frac{\Sigma F x^2}{x_0 \,.\, \Sigma F}$$

The value of x_x, if the axis, x, passes through the center of gravity, G (when $x_0 = 0$) is ∞ which means that:

The center relative to an axis passing through the center of gravity falls on the line lying at an infinite distance away.

In this case, to the pencil having its center in the center of gravity, G, will correspond a range of points lying on the line at ∞, so that it may be said that this line at ∞ is the axis of the center of gravity of the system.

7.—Let G, x, X (Fig. 5), be, respectively, the center of gravity of the system, an axis, and its corresponding center. Take any point, Y, on this axis with y its corresponding axis, which will pass through X; the meeting point, Z, of this axis and the axis, x, will have its corresponding axis, z, passing through X and Y. The points, X, Y, and Z, of this triangle, together with the straight lines, x, y, and z, constitute three pairs of elements which correspond to each other in a double manner; that is, to the center, X, corresponds the line, x, and to the line, x, corresponds the center, X, and, similarly, with the two remaining pairs. The center, Y, and its corresponding axis, y, however, have been chosen arbitrarily, so it may be concluded that the whole system of centers and axes lying on this plane constitutes a polar system, the center of which falls on the center of gravity of this system.

Point X' (the intersection of $G\,X$ with the axis, x) is the center corresponding to the axis through X, parallel to x; X and X' are conjugate centers. If all the forces have the same direction, the distance between a center (pole) and its corresponding axis (polar) is always greater than the distance between the center of gravity of this system and its polar; it follows that, in this case (the only one which is of engineering interest), the auxiliary conic of the system is imaginary. Let X_1 be the symmetrical point, X, with respect to G and imagine the same construction for all the points like X, Y, Z, of the system. Considering all points like X_1 as poles and all axes like x as their corresponding axes, there results a new polar system, the auxiliary conic of which is real and is an ellipse; it is called the "central ellipse of inertia".

Any axis and its corresponding center are polar and anti-pole (that is, symmetrical point of the pole with respect to the center of gravity) with respect to the central ellipse of inertia (auxiliary conic of the polar system). The center (pole) corresponding to a tangent (axis) to this ellipse is the point which is symmetrical to the point of contact of this ellipse. Let x be an axis tangent to the ellipse, and let ρ be the length of the half-diameter conjugate with the direction of x. Taking the moment of inertia, $\Sigma F x^2$, with respect to the axis, x, and computing the distances in the conjugate direction, $x_x = 2\,\rho$, $x_0 = \rho$, and, therefore, on account of Equation (2):

$$\Sigma F x^2 = 2\,\rho \,.\, \rho \,.\, \Sigma F$$

and, therefore, for Equation (1):

$$2\,\rho \,.\, \rho \,.\, \Sigma F = \Sigma F x_0^2 + \rho^2 \,.\, \Sigma F$$

from which:

$$\rho^2 = \frac{\Sigma F x_0^2}{\Sigma F}$$

that is:

The radius of inertia (gyration) corresponding to any axis passing through the center of gravity is given by that half diameter of the ellipse of inertia, the direction of which is conjugate with that of the assumed axis.

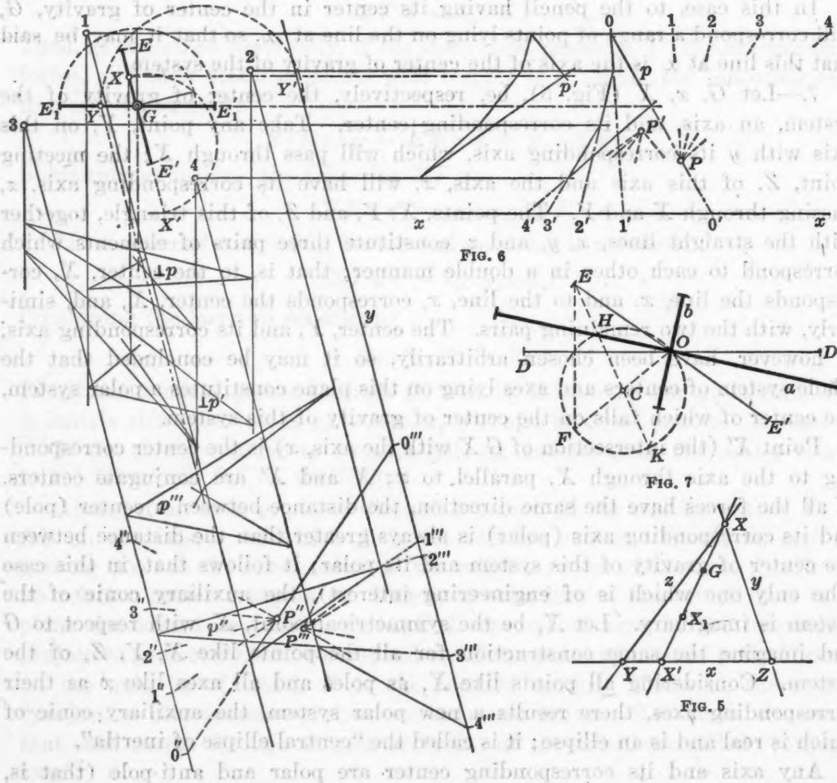

FIG. 6

FIG. 7

FIG. 5

Therefore, the central ellipse of inertia is simply the polar diagram of the law of variation according to which the moment of inertia, with respect to an axis, varies while this axis rotates about the center of gravity of the system. The moment of inertia is maximum and minimum when the axes of the ellipse are, respectively, the minor and major axes; it is for this reason that these two axes are called the "principal axes of inertia" passing through the center of gravity.

8.—Construction of the Central Ellipse of Inertia for a System of Concentrated Forces.—Let 1, 2, 3, and 4 (Fig. 6), be the points of application of four parallel forces and let x be an arbitrary axis. Construct the force line, 0, 1, 2, 3, 4, parallel to the axis, x, with P as a pole. After extending these forces in a direction parallel to the axis, x, connect them by the equilibrium

(funicular) polygon, p, which will give the measure of the static moments $0'1'$, $1'2'$, $2'3'$, $3'4'$, and also the line of action of the resultant force through the center of gravity, G, of the system. Take another pole, P', and connect the static moments just mentioned as new forces applied to the same points, 1, 2, 3, 4, in the same direction as the axis, x. Construct the equilibrium polygon, p', corresponding to this assumption and the resultant line of these static moment-forces will contain X, the anti-pole corresponding to the axis, x. In order to determine the positions of G and X, rotate the given forces through an angle of 90°, also, the static moment-forces, and repeat this same graphical construction. For a rotation of 90°, the two equilibrium polygons will have their sides at 90° with the corresponding sides of the two polygons, p, and p'; it is, therefore, not necessary to construct force polygons, P and P', in this new direction. The lines of resultants given by these two new equilibrium polygons, $\perp p$ and $\perp p'$, will meet the lines of the resultants of the first system at the points, G and X, required. Draw the line, $G X$; the points, X and X', are conjugate, and, therefore,

$$\sqrt{G X \times G X'} = G E$$

which is the length of the semi-diameter of the ellipse of inertia in the direction shown.

Let y be an axis parallel to $G X$; the anti-pole, Y, must fall on the axis passing through G in a direction parallel to x. Lay off $0''$, $1''$, $2''$, $3''$, $4''$, as a line of forces parallel to the axis, y, with P'' as a pole; considering the forces in a direction parallel to y and connecting them with an equilibrium polygon, p'', there is determined on the axis, y, the intercepts, $0'''$, $1'''$, $2'''$, $3'''$, $4'''$, which are proportional to the statical moments of the given forces with respect to this axis. Now, choose any pole, P''', and construct another equilibrium polygon, p''', with respect to the intercepts and the pole just mentioned. The line of resultant thus obtained will meet the axis passing through G and parallel to x at the point, Y. The centers, Y and Y', are conjugate and, therefore,

$$\sqrt{G Y \times G Y'} = G E_1,$$

which is the semi-diameter of the ellipse that is conjugate to the previously found semi-diameter. The lengths and positions of its two conjugate semi-diameters being known, the ellipse is fully determined.

9.—It may be useful to know a method whereby it is possible to determine the axis of an ellipse without drawing the ellipse itself. If $D D'$ and $E E'$ (Fig. 7) are two conjugate diameters of an ellipse, from E draw $E F$ perpendicular to $D D'$ so that $E F = O D$. With C the middle point of $F O$ as a center, pass a circle through F and O. Join C with E, determining the points, H and K; $O H$ and $O K$ are the positions of the axes of the ellipse—the half lengths will be $E K$ and $E H$.*

10.—Given the Polar of an Ellipse to Find its Anti-Pole, and Given the Anti-Pole to Find its Polar.—Let p, Fig. 8, be a polar with respect to an ellipse the axes of which are $A B$ and $C D$. Lay off on $O C$ the segment, $O A' = O A$;

* Chasles, "Section Coniques," Paris, 1865.

similarly, the segment, $O\,C' = O\,C$. Join E with A' and F with C' (D and E being the intersections of p with the y and x-axes, respectively); then, through A' draw $A'\,P_x$ normal to $E\,A'$, and, similarly, through C', obtaining $C'\,P_y$.

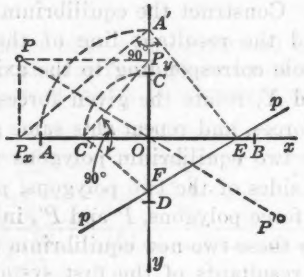

FIG. 8.

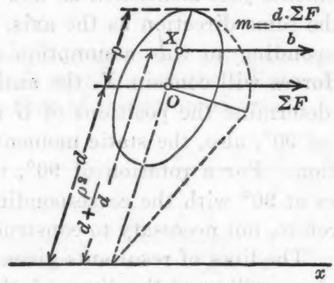

FIG. 9.

The point, P, having $O\,P_x$ and $O\,P_y$ as its co-ordinates, will be the anti-pole of p with respect to the given ellipse. The pole of p will be the point, P', which is symmetrical with P in respect to the center, O.

11.—To find the moment of inertia of a system of co-planar forces with respect to an axis, or the anti-pole of an axis, which are necessary for the construction of the conic of inertia of the system, assume that the given forces are divided into groups. In order to determine the static moments of these forces with respect to the axis, x, substitute the resultant of each one of these groups of forces for the several forces constituting each group, and assume this resultant to be applied to the center of gravity of the group. In fact, if ΣF is the summation of the forces of any one group of the system, and if d is the distance of the center of gravity of this group from the axis, x, $\Sigma F\,x = d\,.\,\Sigma F$. In order to determine the moments of inertia with respect to the same axis, x, or else to determine the position of the anti-pole, X, of the whole system with respect to the axis, x, it is possible to substitute the static moment-forces of the resultants of the various groups for the static moments of the single forces, provided, however, that instead of applying these static moments to the center of gravity of each group, they are applied to the centers of gravity of the static moment-forces of each group, that is, to the anti-poles of the axis, x, with respect to the central ellipses of the various groups (see Fig. 9). The distance between one of these anti-poles and the axis, x, is $d + \frac{\rho^2}{d}$, in which, ρ is the radius of inertia with respect to an axis passing through the center of gravity of that group and parallel to the axis, x. The moment of inertia of that group, therefore, is expressed by:

$$\Sigma F\,x^2 = d\,.\left(d + \frac{\rho^2}{d}\right).\,\Sigma F.$$

Two equilibrium polygons as described in the preceding paragraphs will give graphically the product, $d\,.\,\Sigma F = b\,.\,m$, as well as the other product, $m\left(d + \frac{\rho^2}{d}\right) = c\,.\,n$. For each group of forces, therefore the line of action

of the resultant will act, in the case of the first polygon at a distance, d, from the axis and be applied to the center of gravity of the group; for the second polygon it will act at a distance, $d + \frac{\rho^2}{d}$, from this axis and be applied to the anti-pole of its axis with respect to the ellipse of inertia of that group. This shifting of the line of action is called the "displacement of forces". When for a given group of forces the distance, d, is very large as compared with the radius of inertia, ρ, the displacement, $\frac{\rho^2}{d}$, becomes so small that it may be neglected.

It has been shown how the conception and the generation of the ellipse of inertia of a system of forces by means of graphical methods is possible. In a similar way, the ellipse of inertia of an area may be constructed by substituting for the force or group of forces, an area. The generation of the ellipse of elasticity, therefore, which is nothing other than the ellipse of inertia of some quantities called elastic weights, should by geometrical analogy be already clear.

Before entering into any direct explanation of the theory of the ellipse of elasticity, it is thought expedient to state the following two theorems.

12.—Composition of Rotations.—Let P, Fig. 10, be a point of an elastic system, which, by any means, is moved an infinitesimal distance to P'. The displacement, PP', may be regarded as a rotation about an instantaneous center, say O_1, through an angle, ϕ_1. Let there be another displacement starting from the same point, P, through an angle, ϕ_2, about a center, O_2.

Join O_1 and O_2 by a straight line and erect $A\,P = y$ perpendicular to $O_1\,O_2$. Decompose $P\,P'$ into $P\text{-}P_{y1}$, perpendicular, and $P\text{-}P_{x1}$, parallel to $O_1\,O_2$. Then, the triangles, $A\,P\,O_1$ and $P\,P'\,P_{x1}$, are similar, as their sides are perpendicular. Also, since the angles of rotation are very small, tan . $\phi = \phi$. Hence,

$$\frac{P\,P'}{P\,P_{x1}} = \frac{P\,O_1}{P\,A}$$

but,

$$P\,P' = P\,O_1 \,.\, \tan\phi_1 = P\,O_1 \,.\, \phi_1, \text{ and } \frac{P\,O_1}{P\,P_{x1}}\,\phi_1 = \frac{P\,O_1}{y}$$

Therefore,

$$P\,P_{x1} = \phi_1 \,.\, y$$

Also,

$$\frac{P\,P'}{P\,P_{y1}} = \frac{P\,O_1}{A\,O_1};\quad \frac{\phi_1}{P\,P_{y1}} = \frac{1}{x_1}$$

Therefore,

$$P\,P_{y1} = \phi_1 \,.\, x_1$$

Similarly, for a rotation about O_2 from P:

$$P\,P_{x2} = \phi_2 \,.\, y$$

$$P\,P_{y2} = \phi_2 \,.\, x_2$$

The sums of the components of these displacements will be:

$$P\,P_y = P\,P_{y1} + P\,P_{y2} = \phi_1 \cdot x_1 + \phi_2 \cdot x_2$$

$$P\,P_x = P\,P_{x1} + P\,P_{x2} = \phi_1 \cdot y + \phi_2\, y = (\phi_1 + \phi_2)\, y$$

Let O be the center of gravity of forces proportional to ϕ_1 and ϕ_2 applied, respectively, at O_1 and O_2. If the point, P, rotates about the center, O, through an angle, $\phi = \phi_1 + \phi_2$, the components of this displacement, in the same directions as previously stated, will be given, as proved for the first rotation, by:

$$P_y = \phi \cdot x = \phi_1 \cdot x_1 + \phi_2 \cdot x_2$$

$$P_2 = \phi \cdot y = (\phi_1 + \phi_2)\, y$$

These values are identical with those already found for the sums of the components of the two displacements, therefore: The resultant rotation of two rotations of a point, P, about the centers, O_1 and O_2, respectively, through the angles, ϕ_1 and ϕ_2, is a rotation through an angle, $\phi = \phi_1 + \phi_2$, about the center of gravity, O, of forces proportional to ϕ_1 and ϕ_2, applied, respectively, at the centers, O_1 and O_2.

13.—Principle of Reciprocity.—The theorem of reciprocity by Betti and its corollary by Maxwell may be demonstrated graphically as follows: Let P_1 and P_2 in Fig. 11 be two points belonging to the same plane elastic system. A force, F_1, rigidly connected with P_1, acts on the system, increasing from zero to its maximum value, F_1.

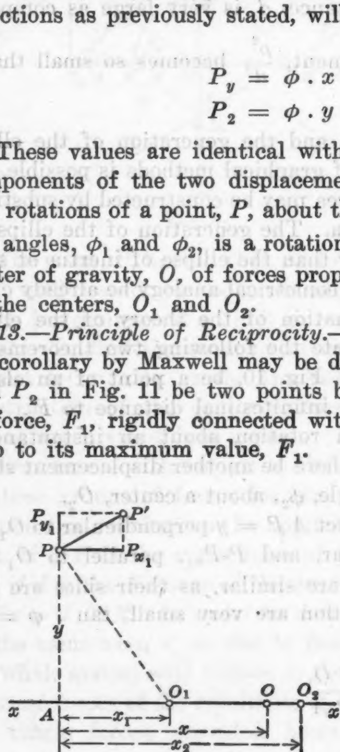

FIG. 10

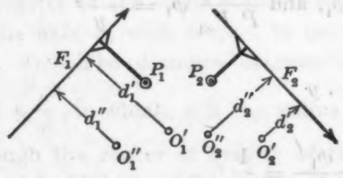

FIG. 11.

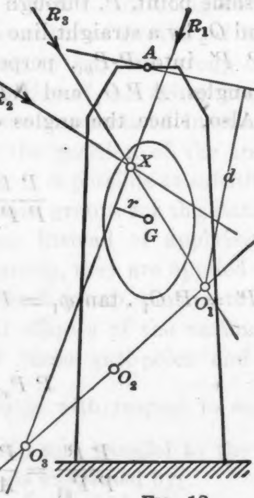

FIG. 12.

The effect on P_1 is to rotate the latter through an angle, ϕ_1' about an instantaneous center of rotation, O_1'. When F_1 reaches its maximum value, a second force, F_2, rigidly connected with P_2, begins to act with an initial value, zero, increasing to its maximum, F_2. A corresponding rotation through an angle, ϕ_2'', about some center, O_2'', results.

The rotation of P_1 caused by F_1 (constant at the value, F_1, and still acting as described), during this interval, will be about some center, O_1'', and will extend through some angle, ϕ_1''. When F_2 has reached its final value, F_2, the virtual work done by both forces on the system, will be:

$$\frac{1}{2} F_1 \cdot \phi_1' \cdot d_1' + \frac{1}{2} F_2 \cdot \phi_2'' \cdot d_2'' + F_1 \cdot \phi_1'' \cdot d_1''$$

If the order of action of F_1 and F_2 on the system is reversed, and if the angle of rotation of P_2 about the new center, O_2', during the action of F_1 is ϕ_2', the virtual work will be:

$$\frac{1}{2} F_2 \cdot \phi_2'' \cdot d_2'' + \frac{1}{2} F_1 \cdot \phi_1' \cdot d_1' + F_2 \cdot \phi_2' \cdot d_2'$$

As, however, the order of action of these two forces is immaterial, as far as the final and initial positions of the elastic system are concerned, the two expressions for the virtual work may be equated, and after cancellation, there finally results:

$$F_1 \cdot \phi_1'' \cdot d_1'' = F_2 \cdot \phi_2' \cdot d_2'$$

which means that:

The virtual work of the force, F_1, during the action of the force, F_2, is the same as the virtual work of the force, F_2, during the action of F_1.

14.—Ellipse of Elasticity.—Let A (Fig. 12) be a terminal point of an elastic body, and R_1 a force rigidly connected to Point A and acting on the whole system. The displacement of Point A caused by the force, R_1, may be regarded as a rotation around a certain point, O_1 (center of instantaneous rotation). If there is another force, R_2, acting on the system similar to R_1, then the displacement of Point A caused by this second force may be regarded as a rotation around another point, O_2 (center of instantaneous rotation). From what has been stated previously, the resultant of R_1 and R_2 will cause a displacement of A the center of instantaneous rotation of which is a point, O_3, lying on the line joining Points O_1 and O_2; that is, if a force rotates about a point, X, the corresponding center of instantaneous rotation moves on a straight line, x.

If O_3 is the center relative to the force, R_3, passing through O_1, then, since during the action of the force, R_1, the force, R_3, does not do any work, by the principle of reciprocity, during the action of the force, R_3, the force, R_1, cannot do any work; therefore, O_3 will fall on R_1, that is:

If a force passes through the center of instantaneous rotation relative to another force, this last force passes through the center of instantaneous rotation relative to the first force.

If a force acts along the straight line, x, its relative center of rotation will fall on X, for since this force passes through O_1, O_3, its relative center of rotation, must fall simultaneously on R_1 and R_3. There are, therefore, three points, X, O_1, and O_3, and three straight lines, x, R_1, and R_3, forming three pairs of elements which correspond to each other in a double manner, namely, to the center, X, corresponds the straight line, x, and to the straight line, x,

corresponds the center, X, and, similarly, for the two remaining pairs. These geometrical relations are true for all straight lines and all points of the plane, and, therefore, they constitute a polar system. The auxiliary conic of this system is imaginary, but considering these lines and all points symmetrical to the centers of rotation with respect to the center of gravity, then there is another polar system, the auxiliary conic of which is real and is an ellipse. With respect to this ellipse, any force and its instantaneous center of rotation are to be considered as polar and anti-pole.

It is this ellipse which is called the "ellipse of elasticity" of a given elastic system, or the ellipse of elasticity of its terminal point, A.

In Fig. 13, G represents the center of gravity of a certain polar system; c represents the auxiliary conic of this polar system; and X_1 and x represent a pair of the infinite number of conjugate pairs of elements of this polar system (X_1 being the pole and x being the polar). It can be seen that the polar passes through the contact points of the two tangents to the conic drawn from Point X_1. X is the anti-pole of x corresponding to Pole X_1 and is taken so that $G\,X_1 = G\,X$.

If a force acts along the line, x, and on the terminal point of an elastic system, and X is the center of instantaneous rotation of this terminal section due to the action of this force, and assuming an infinite number of these lines of action of forces acting on the same terminal section of the system, there will be a corresponding infinite number of centers of instantaneous rotation. All these pairs of elements, like x and X, constitute a polar system, which, for projective reasons, admits only of an imaginary auxiliary conic. If, however, for the infinite points like X, is substituted its symmetrical, X_1, with respect to G, the new polar system constituted by pairs of elements like x and X_1, does admit of a real auxiliary conic, which, for projective reasons, is an ellipse. From the foregoing, it is clear how, having determined the ellipse of elasticity of an elastic system, the center of instantaneous rotation of the terminal point of the system for any force rigidly applied to its terminal point, is readily found.

15.—Elastic Weight.—Let the force, R_1 (Fig. 12), be decomposed into two others, one equal and parallel to R_1, passing through G, the center of the ellipse described, the other infinitely small, with its line of action at infinity (corresponding to a couple). The first component will cause a rotation about the corresponding anti-pole of the diameter on which the component lies. As previously mentioned, however, the anti-pole of a diameter lies at infinity on the conjugate diameter, therefore, this component will cause a displacement of the point on a straight line perpendicular to the conjugate diameter of its line of action. The second component, which is equivalent to a couple the moment of which is $M = R_1 . r$, will cause a rotation through an angle, ϕ, about the center of the ellipse, as this center is the anti-pole of a line at infinity, ϕ being proportional to M, whence: $\phi = G . M$. The constant, G, is called the "elastic weight" of a given elastic system. If this elastic weight, G, is considered as spread all over the construction, and its center of gravity as the center of the ellipse, the following theorems hold good.

16.—Considering the elastic weight, G, as applied at the center of the ellipse and substituting for M its value, $\phi = R \cdot r \cdot G$. Expressed formally this means that:

A terminal point, A, of a plane elastic system acted on by a force, R, rigidly connected with it, will rotate about the anti-pole (with respect to the ellipse of elasticity of the system) of the line of action of R, the angle of rotation being equal to the product of R and the statical moment of the elastic weight G with respect to the line of action of R.

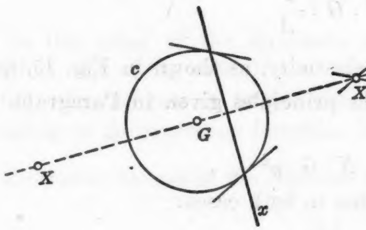

FIG. 13.

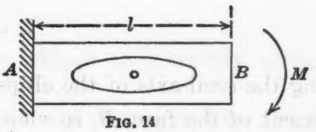

FIG. 14

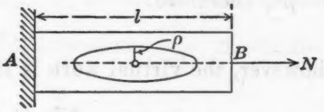

FIG. 15.

17.—The displacement, δ, of the point, A, in a direction, $A\,v$, is equal to the product of the angle of rotation, ϕ, and the distance, d, between the center of rotation and the direction line, $A\,v$. (See Paragraph 12). Whence $\delta = R \cdot r \cdot d \cdot G$. That is:

The displacement of a point, A, in a direction, $A\,v$, due to a force, R, is equal to the force multiplied by the product of inertia of the elastic weight taken with respect to the lines, R and $A\,v$.

18.—If the line, $A\,v$, coincides with the line of action of the force, the product of inertia becomes the moment of inertia taken with respect to the line, $A\,v$.

19.—*Calculation of Elastic Weights.*—Let $A\,B$, Fig. 14, be a fractional part or element of an elastic system acted on by the moment, M. (These elements are to be taken small enough to be considered of constant cross-section, and are to be regarded as fixed at one end and free at the other.) The virtual work done by this moment will be: $L = M\ \phi$, in which, ϕ is the angle of rotation of the terminal section, B, with respect to the section, A. From the theory of deflection:

$$\phi = \frac{M \cdot l}{E \cdot I}$$

Therefore, substituting the value of ϕ given in Paragraph 15,

$$L = M \cdot \phi = M^2 \cdot G = \frac{M^2 \cdot l}{E \cdot I}$$

Hence,

$$G = \frac{l}{E \cdot I}$$

which formula suffices for calculating the elastic weights.

20.—Calculation of Minor Axis of the Ellipse of Elasticity.—Let the element, $A\ B$, Fig. 15, be stretched by a force, N, acting in the geometrical axis of $A\ B$ at the face, B. Its virtual work will be:

$$L = N \,.\, \Delta\, l = N \,.\, \frac{N \,.\, l}{E \,.\, A} = \frac{N^2 \,.\, l}{E \,.\, I} \,.\, \frac{I}{A}$$

in which A = area of cross-section. Substituting the value of G given in Paragraph 19:

$$L = N^2 \,.\, G \,.\, \frac{I}{A}$$

ρ being the semi-axis of the ellipse of elasticity, as shown in Fig. 15, the displacement of the face, B, in view of the principles given in Paragraph 17, is, $N \,.\, G \,.\, \rho^2$; therefore:

$$L = N \,.\, N \,.\, G \,.\, \rho^2$$

As, however, the virtual work is the same in both cases:

$$N^2 \,.\, G \,.\, \frac{I}{A} = N^2 \,.\, G \,.\, \rho^2$$

Therefore:

$$\rho = \sqrt{\frac{I}{A}}$$

which is the semi-axis of the ellipse of elasticity normal to the geometrical axis of the element, $A\,B$.

21.—Calculation of the Major Axis of the Ellipse of Elasticity.—Let $A\ B$ in Fig. 16, be the same element of the system considered previously, and T, perpendicular to the axis, a force passing through the center of the ellipse, and rigidly connected with the face, B. The virtual work becomes:

$$L = T \,.\, \delta$$

which, in the light of Paragraph 17, becomes:

$$L = T \,.\, T \,.\, G \,.\, \rho_1{}^2$$

in which, ρ_1 is the semi-axis shown in Fig. 16.

Dividing the solid, $A\ B$, into elements, $\Delta\, x$,

$$M = T \,.\, x$$

in which, x is the distance between any one of these elements and the center of the ellipse. By the usual formula for virtual work:

$$L = \int \frac{M^2 \,.\, d\, x}{E \,.\, I} + \int \chi\, \frac{T^2 \,.\, d\, x}{S \,.\, A}$$

Integrating between $-\frac{1}{2}\, l$ and $+\frac{1}{2}\, l$, this will become:

$$L = \frac{T^2 \,.\, l^3}{12 \,.\, E \,.\, I} + \chi\, \frac{T^2 \,.\, l}{S \,.\, A} = T\, \frac{T \,.\, l}{E \,.\, I}\left(\frac{l^2}{12} + \frac{E}{S} \,.\, \chi \,.\, \rho^2\right)$$

in which, S is the modulus of elasticity in shear and χ is a numerical constant, depending only on the cross-sectional shape.

From the equation:

$$L = T \,.\, T \,.\, G \,.\, \rho_1^{\,2}$$

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{E}{S}\,\chi\,\rho^2}$$

Assuming that $S = \infty$, as is always permissible in arch design, that is neglecting shear, the preceding formula becomes:

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

which is the value of the semi-axis of the ellipse of elasticity along the geometrical axis of the solid.

21 (*a*).—In the case of lattice work, of the type shown in Fig. 16 (*a*), by substituting in the preceding formulas the area, 2 *A*, of the cross-sections of the upper and lower flanges of the element, and the expression, $\frac{1}{2}$, $A \,.\, h^2$, for I:

$$G = \frac{2\,l}{E \,.\, A \,.\, h^2};\ \rho = \frac{h}{2}\text{*}$$

Furthermore, the displacement caused by a vertical force, 1, is given, according to Paragraph 18, by:

$$1 \,.\, G \,.\, \rho_1^{\,2}$$

On the other hand, the displacement must consist of two terms: The first due to the flanges and derived as the similar term already found for a solid element when $I = \frac{1}{2} A \,.\, h^2$, which term is $\frac{l^3}{6\,E \,.\, A \,.\, h^2}$. The other term is due to the lattice work: the strain is $1 \,.\, \frac{s}{h}$; therefore, the deformation is $\frac{1\,\frac{s}{h} \,.\, s}{E \,.\, A'}$ for each element of the lattice, in which, A' is the area of the cross-section of the element of the lattice and the corresponding vertical displacement is given by $1 \,.\, \frac{s^3}{E \,.\, A' \,.\, h^2}$. Noting that the total number of members of the lattice work is $\frac{l}{f}$,

$$G \,.\, \rho_1^{\,2} = \frac{l^3}{6\,E \,.\, A \,.\, h^2} + \frac{s^3 \,.\, l}{E \,.\, A' \,.\, f \,.\, h^2} = \frac{2\,l}{E \,.\, A \,.\, h^2}\left(\frac{l^2}{12} + \frac{A \,.\, s^3}{2\,A' \,.\, f}\right)$$

Therefore,

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{A \,.\, s^3}{2\,A' \,.\, f}}$$

Finally, in the case of a lattice work of the type shown in Fig. 16 (*b*), to the expression of ρ_1 already given, must be added another term due to the uprights of the lattice work, and this term is derived from the one already

* See Paragraphs 19 and 20.

found for the diagonals by putting A'' in the place of A' and h in the place of s, so that:

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{A \cdot s^3}{2\,A' \cdot f} + \frac{A \cdot h^3}{2\,A'' \cdot f}}$$

As before, disregarding the deformation due to the web members of the lattice, this value becomes:

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

21 (b).—For other types of lattice work, in which the flanges are not parallel, it is generally convenient to consider separately the elastic weights of the single members, and apply them to the corresponding poles (joints).

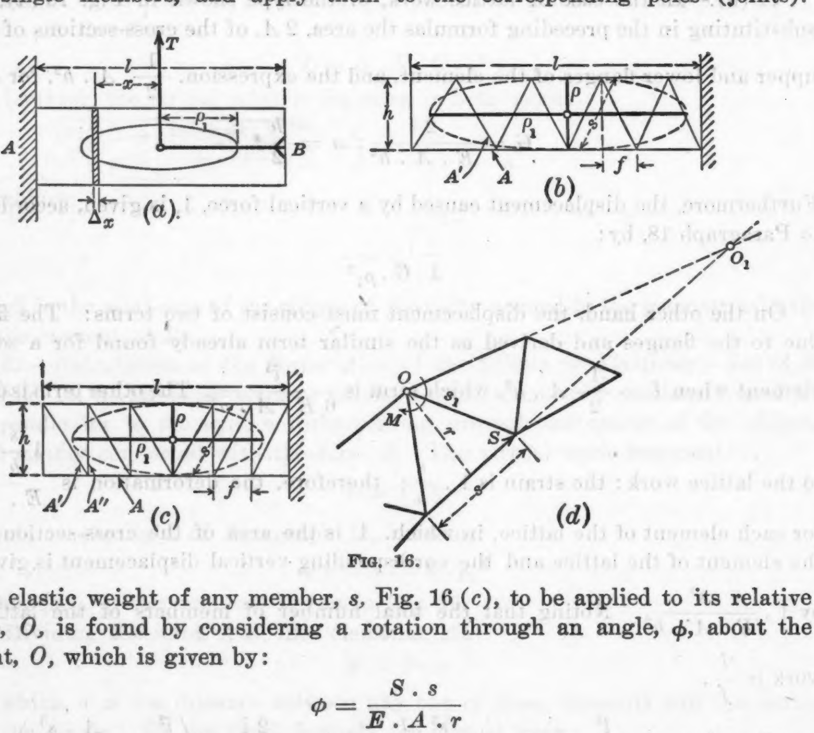

FIG. 16.

The elastic weight of any member, s, Fig. 16 (c), to be applied to its relative pole, O, is found by considering a rotation through an angle, ϕ, about the point, O, which is given by:

$$\phi = \frac{S \cdot s}{E \cdot A \cdot r}$$

in which, S is the strain in the member, s, caused by the moment, M. Noting that $M = S \cdot r$:

$$\Delta\, G = \frac{\phi}{M} = \frac{S \cdot s}{E \cdot A \cdot r} \div S \cdot r$$

that is,

$$\Delta\, G = \frac{s}{E \cdot A \cdot r^2}$$

In the same manner, the elastic weight of a member, s_1, of the web can be determined and this elastic weight should be applied to the pole, O_1. In most

engineering problems it is not necessary to take into consideration the deformations of the web members of a system. The ellipse of inertia of the elastic weights, ΔG, assumed to be applied to the poles of a lattice system, constitutes its ellipse of elasticity.

22.—Elementary Applications.—In the design of an arch, one end is assumed to be free and the other fixed, the ellipse of elasticity with respect to the free end being determined. The elastic weight of each voussoir (calculated as explained in Paragraph 19), considered as a force in accordance with the preceding principles, is applied at the center of gravity of the corresponding voussoir or element into which the arch is divided, and the axes of each elementary ellipse are determined in position and magnitude (see Paragraphs 20 and 21). Then, the ellipse of inertia of these elastic weights, constructed by the same methods used in graphic statics to find the ellipse of inertia, of areas, or of forces, etc., will be the ellipse of elasticity relative to the free end of the system. In other words, the ellipse of elasticity is simply the ellipse of inertia of the elastic weights.

23.—Let $A\,B$ in Fig. 17 be an elastic arch, with a fixed end at B and a free end at A, divided into an arbitrary number of voussoirs. Now, let a force, F, rigidly connected with the terminal point, A, act on the system. It will cause a rotation of the section, A, with respect to the section, B, the value of which will be:

$$\phi = F\,.\,r\,.\,\Sigma\frac{\Delta\,s}{E\,.\,I} = F\,.\,r\,.\,G^{*}$$

in which, $\Delta\,s = l =$ the length of the voussoir, and $\Sigma\dfrac{\Delta\,s}{E\,.\,I} =$ the sum of all the elastic weights, that is, the total elastic weight of the arch, G.

The rotation through the angle, ϕ, will be about D, the anti-pole of F with respect to the ellipse of elasticity of the whole arch (see Paragraph 14).

24.—The displacement, δ, of A along any given direction, $A\,x$, is, by Paragraph 17:

$$\delta = F\,.\,r\,.\,d\,.\,G$$

in which, $d =$ the distance from the direction line, $A\,x$, to the center of rotation, or anti-pole, D, and $r =$ the distance from the center of the ellipse to the line of action of F.

If the arch were fixed at A and free at B, and the force, F, were rigidly connected to Section B, then Section B would rotate, with respect to Section A, through the same angle as before, and about the same point, D.

25.—When the force, F, Fig. 18, passes through the center of gravity, G, of the system, the point, A, moves (Paragraph 6), on the normal, $A\,a$ to y, the conjugate diameter of the line of action of F with respect to the ellipse. It is, therefore, only when a force, F, acts along an axis of the ellipse that the displacement of A is parallel to the line of action of the force (as the axes are the only conjugate diameters of the ellipse perpendicular to each other). The expression for δ, given previously, becomes indeterminate as $d = \infty$ and $r = 0$.

* See Paragraph 16.

The component of the displacement of A in the direction, $A\,x$, however, may be given also by the expression:

$$\delta = F\,.\,r\,.\,d\,.\,G$$

in which, d = the distance from the direction line, $A\,x$, to the center of the ellipse, and r = the distance from the line of action of F to the center of rotation, or anti-pole, X, of the direction line, $A\,x$. (See Paragraph 5.)

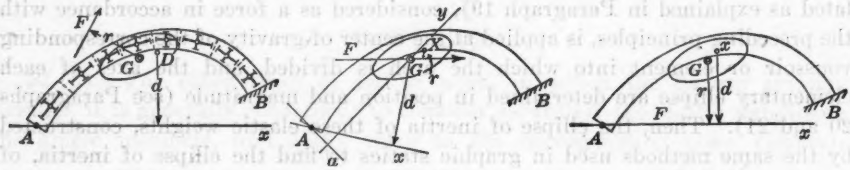

FIG. 17. FIG. 18. FIG. 19.

26.—If F acts along $A\,x$, Fig. 19, the displacement, δ, along this line itself is given by the product of the force, F, and the moment of inertia of the elastic system with respect to the line, $A\,x$; but from Paragraph 6, the expression of δ given in Paragraph 24 remains unchanged, therefore:

$$\delta = F\,.\,r\,.\,d\,.\,G$$

Application to a Trussed Arch Rib

27.—The preceding principles will now be applied to the case of a trussed arch rib, as this is simpler than the case of a solid rib, and as the extension of the general method to the solid arch depends on it. The treatment, however, will be incidental, and only those points of detail that are of interest in the solid arch theory will be carried out in complete working detail.

Fig. 20 represents an arch with fixed ends for which the ellipse of elasticity is to be drawn. Also, there are to be constructed five polygons (funicular polygons).

28.—Polygon p_1.—The elastic weights, w, are treated as vertical forces applied at their respective poles, 1, 2, 3, . . . 11. The eleven elastic weights, w_1, w_2, w_3, . . . w_{11}, are laid off on a vertical, and a pole, P_1, is chosen the distance of which from the vertical is equal to the length of the vertical, that is, $\Sigma\, w$. The funicular polygon, p_1, corresponding to this pole and load line is then laid off to connect the verticals through the poles, 1, 2, 3, . . . 11. The vertical, y (resultant), through the intersection of the first and last sides of the polygon, p_1, will, by ordinary graphics, contain the center of gravity of the system of elastic weights. Furthermore, the sides of the polygon, p_1, produced, will determine on this vertical, segments proportional to the statical moments of the corresponding elastic weights with respect to this same vertical through the center of the system.

A vertical drawn through any pole (the poles or centers of rotation of a truss are the joints)—for example, Pole 4, Polygon p_1—will cut the polygon in two points; the segment, η_0, between these points, is proportional to the statical moment of all the elastic weights to the right of Pole 4 with respect to this vertical.

29.—Polygon p_2.—By the methods of graphic statics determine the axis, x, conjugate with the axis, y (see Paragraph 5 for definition of conjugate axes). At the same time, determine G, the center of gravity of the system of elastic weights at the intersection of the x- and y-axes.* Again, treating the elastic weights as forces applied at the points, 1, 2, 3, . . . 11, this time, however, parallel to the conjugate axis, x, lay off the eleven elastic weights on a line parallel to x, assume a pole, P_2, on y for convenience and, connecting the lines through 1, 2, 3, . . . 11, parallel to x, draw the corresponding funicular polygon, p_2. The center of gravity of elastic weights acting in this direction lies on a line parallel to x through the intersection of the first and last sides of the polygon, p_2. The intersection of this line with the line, y, again gives the center, G, of the system.

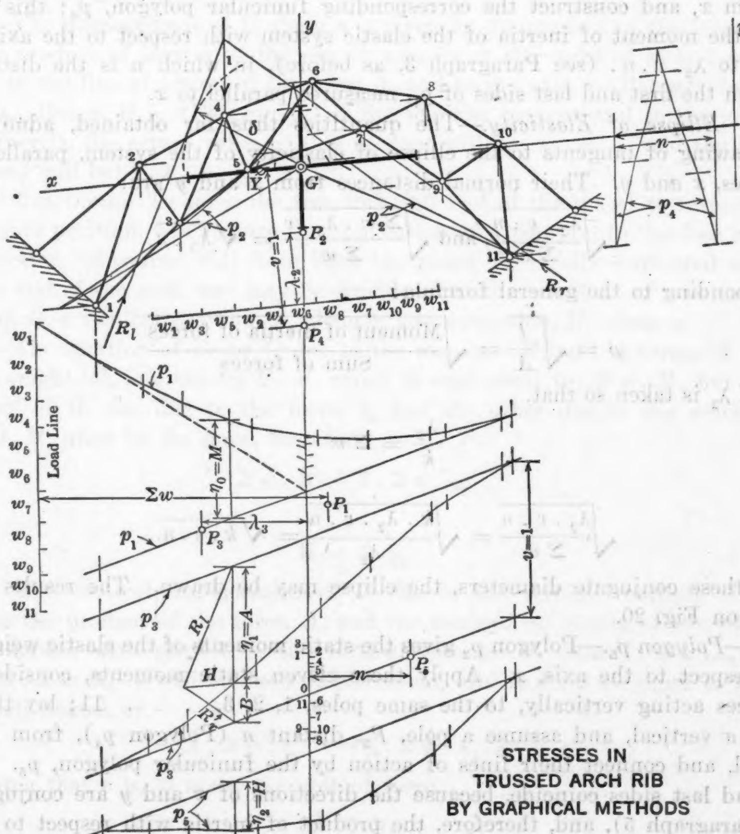

FIG. 20.

30.—Polygon p_3.—Taking a pole, P_3, at an arbitrary distance, λ_3, from y, and considering the statical moments with respect to this vertical, y, as vertical

* In Fig. 20 this construction has been made, assuming any vertical axis, y', and finding its relative pole, Y.

forces applied to the same respective poles, 1, 2, 3, . . . 11, construct the funicular polygon, p_3. Then, the expression, $\Sigma\, w\,.\,\lambda_3\,.\,v$, will be the moment of inertia of the elastic weights with respect to the axis, y, where v is the distance measured parallel to y between the first and last sides of the polygon, p_3. (The values, $\Sigma\, w$, λ_3, and v correspond, respectively, with b, c, and $\Sigma\, n$ of Paragraph 3.)

31.—Polygon p_4.—Just as the sides of p_1 intercept on y segments proportional to the static moments with respect to y of the elastic weights, so the sides of p_2 intercept on x segments proportional to the static moments with respect to the axis, x, of the elastic weights, when they act in the direction parallel to x. Treating such segments as forces parallel to x applied at the poles, 1, 2, 3, . . . 11, previously used, assume a pole, P_4, at a distance, v, from x, and construct the corresponding funicular polygon, p_4; this will make the moment of inertia of the elastic system with respect to the axis, x, equal to $\lambda_2\,.\,v\,.\,n$. (see Paragraph 3, as before), in which n is the distance between the first and last sides of p_4, measured parallel to x.

32.—Ellipse of Elasticity.—The quantities thus far obtained, admit of the drawing of tangents to the ellipse of elasticity of the system, parallel to the axes, x and y. Their normal distances from x and y are:

$$\sqrt{\frac{\lambda_2\,.\,v\,.\,n}{\Sigma\, w}}, \text{ and } \sqrt{\frac{\Sigma\, w\,.\,\lambda_3\,.\,v}{\Sigma\, w}} = \sqrt{\lambda_3\,.\,v}$$

corresponding to the general formula:

$$\rho = \sqrt{\frac{I}{A}} = \sqrt{\frac{\text{Moment of inertia of forces}}{\text{Sum of forces}}}$$

and, if λ_2 is taken so that,

$$\frac{\lambda_2}{k} = \Sigma\, w$$

then,

$$\sqrt{\frac{\lambda_2\,.\,v\,.\,n}{\Sigma\, w}} = \sqrt{\frac{k\,.\,\lambda_2\,.\,v\,.\,n}{\lambda_2}} = \sqrt{k\,.\,v\,.\,n.}$$

From these conjugate diameters, the ellipse may be drawn. The results are shown on Fig. 20.

33.—Polygon p_5.—Polygon p_2 gives the static moments of the elastic weights with respect to the axis, x. Apply these eleven static moments, considered as forces acting vertically, to the same poles, 1, 2, 3, . . . 11; lay them off on a vertical, and assume a pole, P_5, distant n (Polygon p_4), from this vertical, and connect their lines of action by the funicular polygon, p_5. Its first and last sides coincide, because the directions of x and y are conjugate (see Paragraph 5), and, therefore, the product of inertia with respect to the two axes must be zero.

Polygon p_5 will intercept on a vertical through any pole, for example, Pole 4, a segment, η_2, proportional to the product of inertia of elastic weights to the right of Pole 4. Its value is $\lambda_2\,.\,n\,.\,\eta_2$ (see Paragraph 2).

34.—Assuming now a load, 1, to be applied at any point, for example, Pole 4, the left end of the arch being free, and the right end fixed, the force, 1, will dis-

place the free end of the arch and any point rigidly connected therewith, for example, G. Such displacement may be conceived as composed of a rotation, a vertical displacement along y, and a displacement along x.

35.—The rotation of G is equal to the force, 1, multiplied by the sum of the static moments of all the elastic weights to the right of the line of action of the force with respect to this line of action (see Paragraph 16). If η_0 (Polygon p_1) is the segment proportional to the static moment on this vertical, the rotation will be: $1 . \Sigma w . \eta_0$.

36.—The vertical displacement of G is equal to the product of the force, 1, by the moment of inertia of all the elastic weights to the right of Pole 4, with respect to the vertical through Pole 4. If η_1 (Polygon p_3) is the segment proportional to this moment of inertia, then, by Paragraphs 17 and 30, its value is: $1 . \Sigma w . \lambda_3 . \eta_1$.

37.—The displacement of G along x is given by the product of the force, 1, and the product of inertia of the elastic weights at the right of Pole 4 with respect to the line of action of the force, 1, and to the axis, x (Paragraphs 17 and 33). Hence, if η_2 (Polygon p_5) is the segment proportional to the product of inertia, the actual value of this product will be: $\lambda_2 . n . \eta_2$, and the displacement will be: $1 . \lambda_2 . n . \eta_2$.

38.—*Reactions.*—To force the free end (left end of the truss) to return to its original position, will require the application of a reaction to the free end. The reaction, of course, will force back the point, G, rigidly connected with the free end of the arch, and may be considered as the resultant of a moment, M, about G, a vertical reaction, A, along y, and a reaction, H, along x.

39.—The rotation of Point G due to the moment, M, and in terms of the elastic weight, G, is given by G, M, which is equivalent to: $\Sigma w . M$; but the rotations of G, one due to the force, 1, and the other due to the reaction moment, M, must be the same, therefore:

$$\Sigma w . M = 1 . \Sigma w . \eta_0$$

or,

$$M = 1 . \eta_0$$

40.—The vertical displacement of G due to the vertical reaction, A, is equal to the product of the force, A, and the moment of inertia of the whole elastic system with respect to the vertical through G, namely, $\Sigma w . \lambda_3 . v$. Then, $A . \Sigma w . \lambda_3 . v$ will be such displacement, and it must be equal to that previously found, or,

$$A . \Sigma w . \lambda_3 . v = 1 . \Sigma w . \lambda_3 . \eta_1$$

Supposing that v is, to scale, the unit of force,

$$A = \eta_1$$

41.—The displacement of G along x is given by the product of the force, H, and the moment of inertia of the elastic weights of the whole system with respect to the axis, x, that is, by $H . \lambda_2 . v . n$. Putting this equal to the quantity previously found:

$$H . \lambda_2 . v . n = 1 . \lambda_2 . n . \eta_2$$

Assuming, again, that v is the unit of force,

$$H = \eta_2$$

42.—Influence Lines.—It is evident, therefore, that the three polygons, p_1, p_3, and p_5, are the influence lines of the three quantities M, A, and H.

43.—Position of Reactions.—The first and last sides of Polygon p_3 are extended intercepting on the vertical through the load segments which are equal to the vertical reactions at the supports (Fig. 20). It is then easy to find the reactions, R_l and R_r, by combining the vertical components with the horizontal thrust already found, as shown on Fig. 20.

To find the actual position of the two reactions with respect to the arch, the moment, M, of the force acting is given with respect to the center, G, hence:

$$M = 1 \cdot \eta_0 = R_l \cdot \delta$$

in which, δ is the lever arm of the moment, therefore:

$$\delta = \eta_0 \cdot \frac{1}{R_l}$$

Hence, a tangent parallel to R_l obtained from Polygon p_3 to a circle of radius, δ, with center at G, will be the line of action of R_l. The other reaction, R_r, may be obtained similarly, or, alternatively, by proceeding from the fact that the line of action of R_r must pass through the intersection of the vertical of the load, 1, and the line of action of R_l, and be parallel to R_r in Polygon p_3. These two lines of action may also be obtained by other methods.

44.—Intersection and Envelope Lines.—The geometrical locus of the intersections of R_l and R_r is called the "line of intersection"; the curve tangent to the left reactions is the "left envelope line", and the similar curve on the right is the "right envelope line."

Application to a Solid Arch Rib

45.—As far as deformation is concerned, a latticed rib may be substituted for a solid rib, as will become clear from the following demonstration:

Consider the solid rib to be divided into elementary lengths, $d\,s$, with N and M, respectively, a normal force and its moment with respect to the middle cross-section of one of these elements. N is the resultant of the external forces applied to the arch to the left, for example, of the section considered. The shear, as is customary, is neglected as being very small for arches.

The deformation, $\epsilon \,.\, d\,s$, due to the elasticity of the material and temperature changes, is given by the equation:

$$\varepsilon \,.\, d\,s = \frac{N \,.\, d\,s}{E \,.\, A} + a \,.\, t \,.\, d\,s$$

and the angle, $d\,\theta$, of rotation of the terminal section of the element with respect to its first section is:

$$d\,\theta = \frac{M \,.\, d\,s}{E \,.\, I}$$

in which,

N = the normal force;
E = the modulus of elasticity;
A = the area of cross-section;
a = the coefficient of expansion;
t = the number of degrees of temperature change;
M = the moment; and
I = the moment of inertia of cross-section.

Imagine $u_1\ u_2$ and $v_1\ v_2$, Fig. 21, to be the geometrical axes of the upper and lower flanges of a certain hypothetical latticed arch, all material of the solid section being assumed to be concentrated in these flanges. Furthermore, $u_1\ u_2$ and $v_1\ v_2$ are assumed to be in the same plane with the force, and parallel to the geometrical axis of the element considered. Let $\rho = \sqrt{\frac{I}{A}}$ be their distance from the axis; also, let v be the middle point of $v_1\ v_2$, which will be assumed to be the pole of $u_1\ u_2$, so that the lattice shall consist of $u_1\ v$, $v\ u_2$; similarly for the point, u.

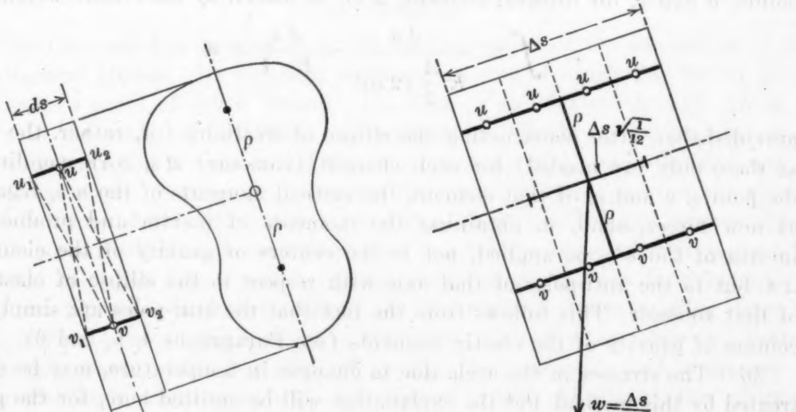

FIG. 21.

The stresses, therefore, in the flanges, $u_1\ u_2$, $v_1\ v_2$, due to the force, N, and the moment, M, with respect to the middle section of the element, $d\,s$, will be:

$$-\frac{N\rho + M}{2\rho} = -\frac{N}{2} - \frac{M}{2\rho}$$

$$-\frac{N\rho - M}{2\rho} = -\frac{N}{2} + \frac{M}{2\rho}$$

and the deformation of the panel, $u_1\ u_2$, $v_2\ v_1$, of the hypothetical latticed arch, considering only the relative displacement of the terminal sections of the element, $d\,s$, will be the same as for the solid element. In fact, due to the normal force, N, and the change of temperature, t^0, the points, u_2 and v_2, will be dis-

placed with respect to the points, u_1 and v_1, in a direction parallel to the geometrical axis of the element, $d\,s$, and such displacement will be equal to:

$$-\frac{\frac{N}{2}\,.\,d\,s}{E\,\frac{A}{2}} + a\,.\,t^0\,.\,d\,s = -\frac{N\,.\,d\,s}{E\,.\,A} + a\,.\,t^0\,.\,d\,s = \varepsilon\,.\,d\,s$$

and due to the moment, M, the line, $u_2\,v_2$, will rotate with respect to the line, $u_1\,v_1$, about its middle point through an angle given by:

$$\frac{1}{\rho}\,.\,\frac{\frac{M}{2\,\rho}\,.\,d\,s}{E\,.\,\frac{A}{2}} = \frac{M\,.\,d\,s}{E\,.\,I} = d\,\theta$$

so that the evaluation of the statically indeterminate quantities and the deformations of the elastic arch with solid cross-section is always possible by the substitution of the upper and lower flanges of a hypothetical latticed arch.

Moreover, it is legitimate to combine sufficient infinitesimal elements, $d\,s$, to obtain a sum, $\Delta\,s$, such that it may be regarded as a prismatic solid (Fig. 21), and then consider the centers of gravity of these elements, $\Delta\,s$ (centers of the points, u and v, for integral element, $\Delta\,s$), as loaded by the elastic weights:

$$\int 2\,.\,\frac{d\,s}{E\,\frac{A}{2}\,(2\,\rho)^2} = \frac{\Delta\,s}{E\,.\,I} = w$$

provided that, after constructing the ellipse of elasticity (or, rather, the axes, as these only are needed) for each element (voussoir) $\Delta\,s$, corresponding to the points, u and v, of that element, the statical moments of the w's, regarded as new forces, shall, in obtaining the moments of inertia and products of inertia of the w's, be applied, not to the centers of gravity of the elements, $\Delta\,s$, but to the anti-poles of that axis with respect to the ellipse of elasticity of that voussoir. This follows from the fact that the anti-poles are simply the centers of gravity of the elastic moments (see Paragraphs 4, 5, and 6).

46.—The stresses in the arch, due to changes in temperature, may be easily treated by this method, but the explanation will be omitted here, for the paper presented has not dealt with this part of the design.

THE COLORADO RIVER PROBLEM*

By William Kelly,† M. Am. Soc. C. E.

Synopsis

So much has been written and spoken on the subject of the Colorado River in the past few years that perhaps the greatest need at present is an impartial digest of the facts and an estimate of the probabilities. This paper attempts to give both; it also presents some as yet unpublished studies of flood protection, water supply, irrigable areas, and power demand which are important factors in any study of the Colorado. Finally, it sets forth the salient features of a comprehensive scheme of development and points out the lines on which activities of the Federal Government should be directed.

Introduction

The Colorado has its sources in the melting snows of the mountains of the Continental Divide. Its drainage basin has an area of nearly 250 000 sq. miles and covers parts of seven States. Its delta is in Mexico through which it flows to the Gulf of California. Development, mostly irrigation, has gone forward up to the present with little or no thought of a comprehensive use of the entire river. Fortunately, what has been done will not interfere materially with full use of this great water resource, but in view of the large projects under consideration it is important that a general scheme of development be adopted and that supervision be exercised to require future developments to conform thereto. Such supervision can be exercised under existing laws by co-operation between the Federal Government and the seven interested States. Although many additional data on stream flow, location, and character of irrigable lands, dam sites, etc., are needed before details of the ultimate development can be determined, sufficient information is available to determine the general scheme insofar as that is necessary for the consideration of projects now advocated.

Scheme of Development

The essential features of a comprehensive plan of development for the Colorado are determined by topographic conditions. The Upper Basin, which lies above the junction of the main river with the Green River in Utah, has possibilities of development for both irrigation and power; the total area that can be irrigated probably does not exceed 4 500 000 acres, of which about one-

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† Col., Corps of Engrs., U. S. A.; Chf. Engr., Federal Power Comm., Washington, D. C.

third is now irrigated. This area is from 4 000 to 8 000 ft. above sea level and use of water for irrigation will always be limited by climatic conditions. The depletion of water supply by irrigation in this area will probably never exceed 4 500 000 acre-ft. To this should be added diversions out of the Basin which may reach 500 000 acre-ft.

In the Upper Basin there are many possibilities for power development, the most promising of which are on the Green between Green River, Wyo., and the junction of the Green with the Colorado. Interference between these power developments and irrigation should be given careful consideration before the developments are undertaken. Such consideration is assured by the fact that practically all such projects involve the use of public lands and thereby come under the jurisdiction of the Federal Power Commission.

The Middle or Canyon Section extends from the junction of the Green to a short distance below Needles, Calif. Irrigation in this section will never be important, being limited to a few inter-canyon valleys of doubtful feasibility. The importance of this section is due to the 4 000 000 h. p. it can produce. If water is reserved for up-stream irrigation, and re-regulation is provided for irrigation below, there will be no interference with irrigation by power in this section.

Below Needles, including Mexico, there are more lands that can eventually be irrigated than there will be water to serve. Power possibilities in this section are small and will be incidental to irrigation development. The Gila River enters the Colorado just above Yuma, Ariz. The development of its water resources has no bearing on the scheme for the Colorado, except in the matter of flood protection of the delta region.

Permanent settlement of the Colorado Basin will be largely dependent on irrigation. It is probable that in the immediate future power will have a greater value than irrigation, but as power can be obtained from other sources than the river, it should not be allowed to curtail the ultimate irrigation development.

Although it is impossible to predict the rate at which either irrigation or power will develop, it is certain that there are more than enough irrigable lands to use all the water available and that all the power that can be developed in the main section of the river will be needed. The scheme of development, therefore, should provide that:

1.—Losses from evaporation should be kept to a minimum.

2.—All available head should be used for power.

3.—Storage for regulation of flow should be located above the Canyon Section so that the equated flow can be used through the greatest practicable head.

4.—Storage in and below the Canyon Section should be limited to that necessary for re-regulation of flow for irrigation in the Lower Basin plus such quantity as is essential for immediate flood relief of existing developments in the Lower Basin. Failure to conform to this provision will mean ultimate duplication of storage capacity and consequent curtailment of irrigation due to unnecessary evaporation losses.

There are reservoir sites in the Upper Basin of sufficient capacity to provide regulation of flow. The first cost of such reservoirs per acre-foot of storage will be more than that for sites in the lower Canyon Section, but they will be worth as much for irrigation, three times as much for power, and, on account of regulated flow, will make possible material savings in cost of power dams built below them. Moreover, they can be developed successively as needed so that if interest on investment is considered, their ultimate cost may not be much greater, even on an acre-foot basis.

Present developments in the Lower Basin are subject to damage by floods. The development of storage in interest of power will relieve the flood menace in part. Full protection, insofar as it can be obtained by storage, requires that the storage capacity be available at the beginning of each flood season. Storage for power on the other hand should be operated to keep reservoirs as full as practicable at all times. The United States Bureau of Reclamation estimates that ultimately 4 000 000 acre-ft. will suffice for flood protection. As will be shown hereafter, 4 000 000 acre-ft. is all that is justified at present. Flood storage to afford the greatest benefit should be as far down stream as practicable.

Flood Protection

The Lower Basin is menaced by floods from both the Gila and the Colorado Rivers. The Gila floods are produced by winter rains and generally occur between November 30 and March 1. They reach a maximum flow of about 200 000 sec-ft. (approximately the flow over Niagara Falls). They are extremely flashy and, therefore, produce higher flood stages and greater velocities than the more deliberate floods of the Colorado. As long as the Gila remains uncontrolled, it will determine the height to which levees must be maintained at and below Yuma.

There is general agreement that, although the Gila, until controlled, will require maintenance of levees at present heights, it will never seriously inundate the Imperial Valley, because it discharges no water for a large part of the year. There are several irrigation and power projects under consideration, which, if completed, will materially reduce the magnitude of Gila floods. No proposal is being seriously advanced that the Federal Government should undertake control of the floods of the Gila River.

The main floods in the Colorado are due to melting snow in the upper regions of the river. About 86% of the total run-off at Yuma comes from north of Arizona. These floods begin as early as March and may last into July, and their maximum flow is about the same as that of the Gila, but they contain vastly more water. The floods in the Lower Colorado affect three existing projects, namely, Palo Verde, Yuma, and Imperial Valley, both in the United States and in Mexico.

Palo Verde.—The Palo Verde Project (see Fig. 1) is on the west side of the river in Riverside and Imperial Counties, California. The Project is protected by a levee 28½ miles long. The engineer of the Project states that the lands would be flooded if no levees were provided when the flow exceeds 50 000

sec-ft. and that bank protection is required when the flow exceeds 35 000 sec-ft. The total area irrigable is 78 000 acres, of which 44 000 acres were irrigated in 1922.

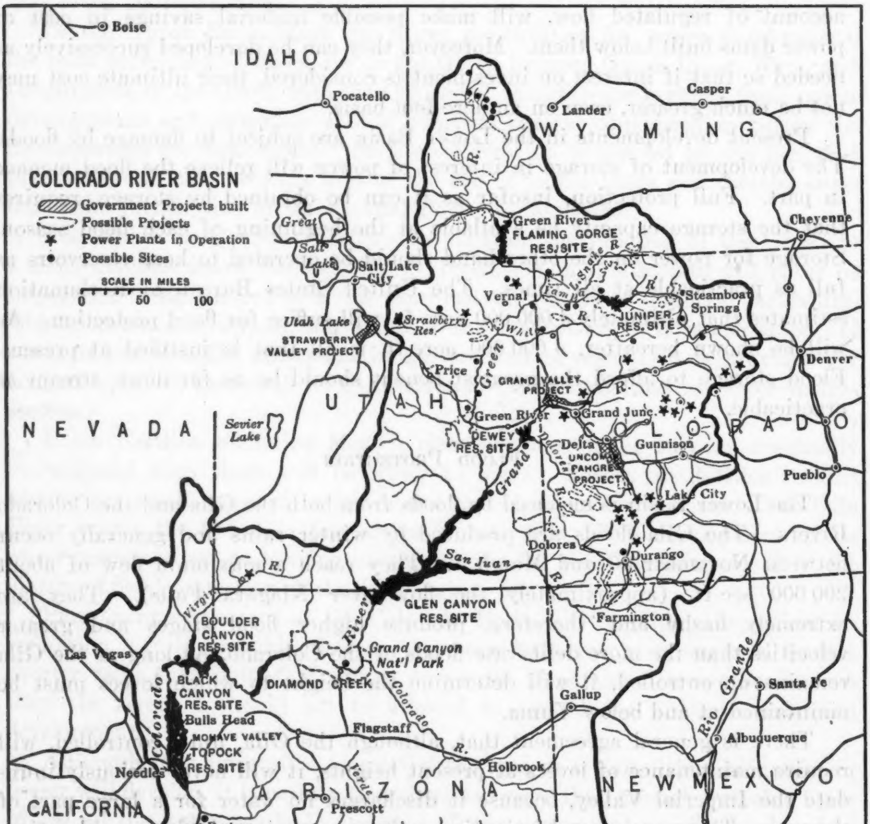

FIG. 1.

Yuma.—The Yuma Project (see Fig. 2) has a total irrigable area of 114 000 acres, 55 700 acres of which were irrigated in 1922. Of this land, 15 000 acres lie in the Yuma Indian Reservation on the California side between Laguna

Dam and a point opposite Yuma. These lands were subject to overflow at extreme high water and have been protected by a levee about 12 miles long, extending from Laguna Heading to the Southern Pacific Railroad track near Yuma. The Potholes Branch of the Southern Pacific Railroad is located on this levee. The levee crosses several old meanders of the river and, consequently, is subject to attack by it at nearly every high water. Bank protection has been necessary to maintain the levee, and the bank is now revetted practically throughout its entire length. The levee has been breached by the Colorado River floods once (in 1913) and was overtopped at its lower end by the Gila flood of January, 1916.

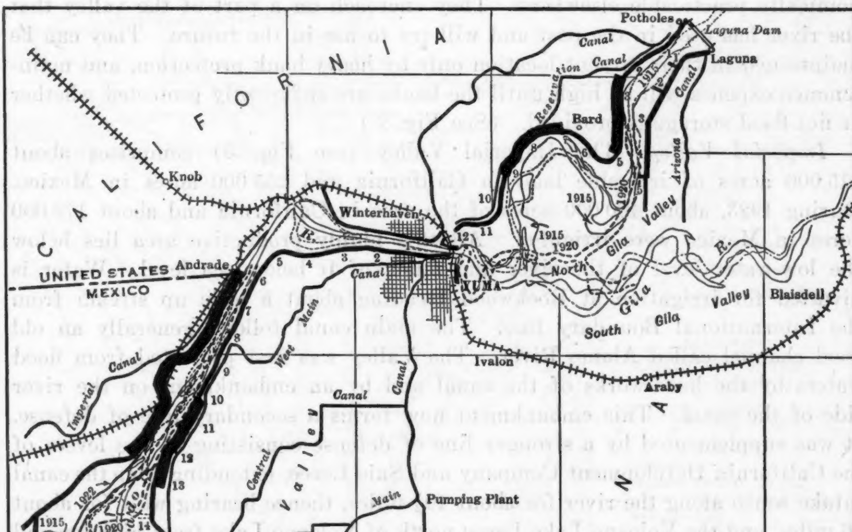

FIG. 2.

Another 54 000 acres lie in Yuma Valley on the Arizona side between the Town of Yuma and the Mexican Boundary. These lands were subject to overflow at high water and have been protected by a levee 25 miles long, extending from Yuma to the Mexican Boundary. A Government railroad has been built on this levee in order to facilitate its maintenance. For 12 miles below Yuma, the river flows in a narrow and relatively straight channel with fairly stable banks and has never given much flood trouble. Below that point, it shows a

determination to repeat its meandering into the land now occupied by the Project. Since 1909, the river has made direct attacks on this levee at 12, 16, and 24 miles below Yuma, necessitating heavy expenditure for bank protection. The levee has never been overtopped, but has twice failed from undercutting, once from the Gila flood of January, 1916, and once from the Colorado River flood of 1920. The construction and maintenance of the levees of the Yuma Project cost \$3 234 470. The annual maintenance for the past six years has averaged \$86 420. The remainder of the lands in the Project, about 45 000 acres, are on the mesa east of Yuma Valley, well above high water.

The Yuma levees are nearer the river than has been found to be economically practicable elsewhere. They encroach on a part of the valley that the river has used in the past and will try to use in the future. They can be maintained in their present location only by heavy bank protection, and maintenance expense will be high until the banks are sufficiently protected whether or not flood storage is provided. (See Fig. 2.)

Imperial Valley.—The Imperial Valley (see Fig. 3) comprises about 515 000 acres of irrigable land in California and 255 000 acres in Mexico. During 1923, about 400 000 acres of the area in California and about 170 000 acres in Mexico were irrigated. All this highly productive area lies below the low-water level of the river and much of it below sea level. Water is diverted for irrigation at Rockwood Heading about a mile up stream from the International Boundary line. The main canal follows generally an old flood channel called Alamo River. The Valley was first protected from flood waters by the head-works of the canal and by an embankment on the river side of the canal. This embankment now forms a secondary line of defense. It was supplemented by a stronger line of defense consisting of the levees of the California Development Company and Sais Levee, extending from the canal intake south along the river for about 10½ miles, thence bearing west for about 16 miles, and the Volcano Lake Levee north of Volcano Lake from high ground west of the Valley to the Inter-California Railway Company embankment and the Imperial Canal near Bataques Station.

In 1905, the Imperial Canal had silted, for about 4 miles below the Heading, to such an extent as to threaten a water famine during the low-water period. In order to get water into the canal, a cut was dredged to turn the river into it at a point about 7 miles below the Heading. An early flood season defeated attempts to close the dredged cut and the entire flood flow passed into the Valley. It was not until 1907 that efforts to turn the river back into its old channel were successful. The closing of this break cost about \$2 000 000. In 1909, the river left its old channel at a point about 29 miles below Yuma and turned west through Bee River into Volcano Lake. This necessitated the extending of the Sais Levee to an intersection with the Volcano Lake Levee and of raising and reinforcing the Volcano Lake Levee. Almost the entire silt load of the river was deposited in Volcano Lake and its bed was raised at a rate that made it difficult to keep the Volcano Lake Levee above it. In 1911, the Federal Government provided \$1 000 000 to relieve the situation. The funds were used to build the Ockerson Levee in an attempt to put the river back into its old channel. (See Fig. 3.) The attempt was unsuccessful. In 1922,

the Pescadero Cut-Off was built and the river successfully turned from Bee River and Volcano Lake into Pescadero River. As a part of the cut-off project, the Ockerson Levee was repaired and extended about 8 miles along the north bank of Bee River, thence south across the river and down the west bank of the cut-off channel. This new levee now constitutes the main line of defense. No alarm is now felt for the levee of the California Development Company from the intake down to the Ockerson Levee, because, in its present condition, except during Gila floods, the river never gets over its banks along this stretch. Even

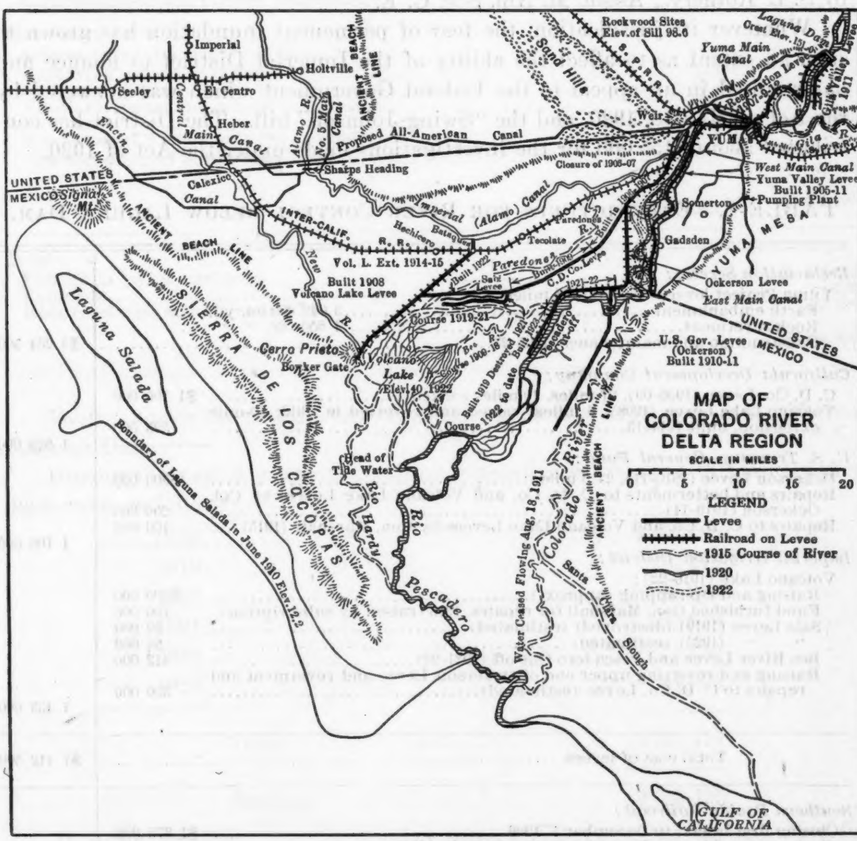

FIG. 3.

if the levee were breached, as it was in 1914 and again in 1916, there would be no flow into the valley. If the new levee should fail, however, the river would return to Volcano Lake and again threaten the Valley with inundation over the Volcano Lake Levee. The new levee is a substantial structure with a railroad throughout its full length and rock revetment on the river side. During the long flood season, however, the ground under and on both sides of the levee becomes saturated and softened so that, if the river in its meanders should start a direct attack, the levee might fail by undercutting in spite of all efforts to save it. Between $6 000 000 and $7 000 000 has been spent on the flood protection of Imperial Valley, of which amount the Southern Pacific

Railroad Company has paid about $2 300 000 and the Federal Government, $1 100 000. (See Table 1.) The annual maintenance is said to be from $200 000 to $600 000.

A study of the reports on flood protection of the Imperial Valley shows that the engineers engaged on the work have never been alarmed over the danger of serious inundation of the Valley since the river was shut out in 1907. In this connection attention is invited to the papers by H. T. Cory,* M. Am. Soc. C. E., who closed the gap in 1907, J. C. Allison,† M. Am. Soc. C. E., and by S. L. Rothery,‡ Assoc. M. Am. Soc. C. E.

Whatever its justification, the fear of permanent inundation has grown to such an extent as to affect the ability of the Imperial District to finance and has resulted in an appeal to the Federal Government which was answered by the Act of May 18, 1920, and the "Swing-Johnson" bill. The District has contributed about $155 000 for the investigations made under the Act of 1920.

TABLE 1.—EXPENDITURES FOR RIVER CONTROL BELOW LAGUNA DAM.

Reclamation Service:		
Yuma Project levees (1905-12), 54 miles:		
Earth embankment....3 040 000 cu. yd.		
Rock revetment....1 650 000 " "		
Construction and maintenance, 1905-23....		$3 234 500
California Development Company:		
C. D. Co. Levee (1906-09), 27 miles, 10 miles rocked....	$1 100 000	
Volcano Lake Levee (1908), 8 miles; raised and revetted in 1912; 10-mile extension, built 1914-15....	525 000	
		1 625 000
U. S. Treasury—General Fund:		
Ockerson Levee (1910-11). 24.5 miles:....	$800 000	
Repairs and betterments to C. D. Co. and Volcano Lake Levees by Col. Ockerson (1910-11)....	200 000	
Repairs to C. D. Co. and Volcano Lake Levees by Gen. Marshall (1915)....	100 000	
		1 100 000
Imperial Irrigation District:		
Volcano Lake (1916-22):		
Raising and rip-rapping (approx.)....	$500 000	
Fund furnished Gen. Marshall for repairs, 1915 (raised by subscription)..	100 000	
Sais Levee (1919) (destroyed) (estimated)....	60 000	
" " (1922) (estimated)....	80 000	
Bee River Levee and Pescadero Cut-off (1921-22)....	413 000	
Raising and revetting upper end of Ockerson Levee and revetment and repairs to C. D. Co. Levee (estimated)....	300 000	
		1 453 000
Total cost of levees....		$7 412 500
Southern Pacific Railroad:		
Closing first break, to December 1, 1906....	$1 375 000	
Closing second break, December 7, 1906-July 21, 1907....	1 084 000	
		$2 459 000
Total for river control....		$9 871 500

Average annual cost of maintenance for past 6 years:
Yuma Project.......$ 86 500
Imperial Valley......200 000

* "Irrigation and River Control in the Colorado River Delta", *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1204.

† "Control of the Colorado River as Related to the Protection of Imperial Valley", *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 297.

‡ "A River Diversion on the Delta of the Colorado in Relation to Imperial Valley, California", *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1412.

TABLE 1.—(*Continued.*)

EXPENDITURES ON LEVEES, YUMA PROJECT.

Yuma Valley Levee (built in 1905–11):		
1 125 969 cu. yd. of earthwork	$ 353 608	
26.5 miles of railroad	231 889	
Rock revetment and general maintenance	1 142 640	
		$1 728 137
Reservation Levee (built 1907–09 and 1912):		
1 166 404 cu. yd. of earthwork	$ 335 252	
Railroad	5 670	
Rock revetment and general maintenance	761 290	
		1 102 212
Gila Valley Levees:		
Arizona Levee (5½ miles built 1911):		
402 005 cu. yd. of earthwork	$ 84 996	
6.5 miles railroad	71 423	
Rock revetment and general maintenance	184 648	
		341 067
Gila Levees (7.8 miles built 1906; abandoned):		
339 804 cu. yd. of earthwork	$ 63 053	
		63 053
		$3 234 469
Totals:		
3 034 182 cu. yd. of earthwork		$ 836 909
Railroads		308 982
Rock revetment and maintenance		2 088 578
		$3 234 469

Table 1 shows the expenditures that have been made for river control below Laguna Dam.

During the past 6 years, the annual expenditures charged to maintenance have been as follows:

1918	$ 65 789
1919	73 471
1920	86 609
1921	122 958
1922	74 854
1923	94 835
Total	$518 516
Average	$ 86 420

It is not possible from the data at hand to segregate the expenditures for rock revetment and maintenance prior to 1918 into amounts for current maintenance and for permanent improvements. Some of the rock has been used to slow down erosion of natural banks at dangerous bends that were rapidly approaching the levee, and much that has been placed in concentrated quantities on the levee at temporary danger points should not be classed as permanent improvement. Of the rock, 1 650 000 cu. yd. have been placed at a cost of approximately $1 600 000, or about $1 per cu. yd. Of this amount, about three-fourths, or $1 200 000, should be considered, it is believed, as chargeable to permanent betterments. This leaves $888 578 as the current

maintenance cost for the period, 1909–23, and an average of about $67 000 per year for the 13 years of active defense of the entire system.

Flood Relief Required

The difficulties in maintaining the levees in the Lower Colorado Basin have resulted in an effort to seek relief by providing storage to reduce the flood flow.

There is considerable difference of opinion as to what flow can be handled with safety in the Lower Colorado. This is not surprising because of the wide difference in flood height for any given flow. The Yuma gauge has recorded a stage of 25 ft., with flows ranging from 35 000 to 121 000 sec-ft. In a river with a stable channel, flood heights can be predicted for any given flow, but the Colorado is not such a river. It flows in a deep bed of silt and the channel enlarges as the flow increases, so that the effect of reducing the flow by storage is uncertain. The best information available on the Lower Colorado is contained in records and reports of the U. S. Bureau of Reclamation. The most accurate data regarding flow, velocity, silt content, and scour come from the Yuma gauging station and cannot be generally applied to the entire river. They do apply with fair accuracy, however, to the river from Laguna to the delta cone.

E. C. Bebb, Assoc. M. Am. Soc. C. E., has collected and compiled most of the information on which the following conclusions are based. The ability of the river to pass its flood flow without damage to the Lower Basin depends on three factors, namely, (*a*) the quantity of water that has to be passed and the rate at which it passes; (*b*) the amount of silt carried through and deposited in the lower reaches; and (*c*) the amount of scour performed in the bed of silt through which the river flows.

The danger of breaching the levees is rarely from pressure of high water, but from undercutting due to meandering of the stream. The tendency to meander is present at all stages, but is particularly serious during a falling stage when the banks are saturated and sloughing is accelerated by reduction of hydrostatic pressure from the river. During this stage, also, meandering is accentuated because, with the checking of velocity, the heavier silt deposits rapidly in the slower water near the convex bank at bends, thus contracting the channel and deflecting the main current more and more toward the concave bank. If at this stage a small quick rise occurs that rapidly increases the velocity without giving time for the removal of the newly formed bar, the saturated bank will be cut away rapidly. The records indicate that undercutting is likely to proceed more rapidly and to a greater depth the greater the quantity of water flowing in the river and, therefore, may get beyond control when the flow is very large.

The silt brought down by the river affects the flood problem in three ways:

1.—It accentuates the meandering of the stream and the instability of the channel as just described.

2.—It deposits in the delta and gradually raises the bed of the river, thereby requiring higher levees. This process is slow unless the

deposit is concentrated over a small area near a levee as happened with the Volcano Lake Levee from 1909 to 1922.

3.—It may fill the flood channel excessively. Winter floods from the lower tributaries, including the Gila, carry excessive amounts of sediment, much of which deposits in the flood channel. In years when late summer and winter floods are above normal, the flood channel is filled to a point that may cause trouble when the large summer flood starts. This is shown by comparing the gauge heights at a flow of 10 000 sec-ft. (Fig. 4) with the hydrograph (Plate V).

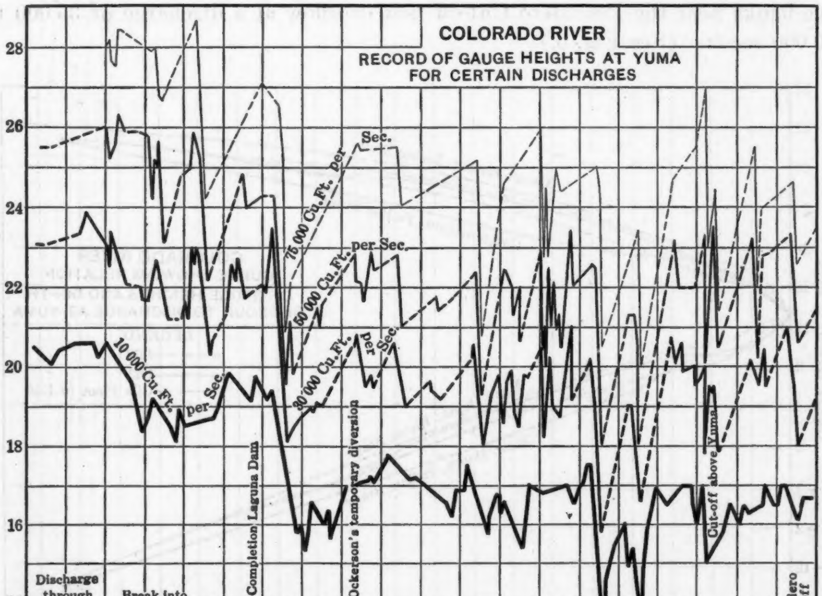

FIG. 4.

From Laguna Dam to Yuma, the river flows through a broad valley and has all the characteristics of a meandering alluvial stream. At Yuma, it is contracted by solid banks to a width of less than 600 ft. and continues in a channel not more than 1 000 ft. wide for about 9 miles. This section of the river has never given serious flood trouble. The upper end of the old delta begins at this point, but in its present condition the river does not take the characteristics of a stream in its delta until it reaches the Pescadero Cut-off about 30 miles below. In the stretch from 9 to 40 miles below Yuma, the river now meanders with about the same characteristics as it does above Yuma. Just below the Pescadero Cut-off, the river spreads on its present delta cone. The area covered by the spread is relatively small and the accretions are correspondingly rapid. As the cone builds up, the delta condition probably will

extend up stream so that the banks will eventually overflow at high water for some distance above the Pescadero Cut-off no matter how much the maximum flood is reduced. The normal slope of the Colorado is 1.1 ft. per mile. Under present conditions, when the Yuma gauge is below 25.4 ft., the river does not ordinarily get over its banks on the Yuma Project from Laguna to Mile 16 on the levee below Yuma, nor on the Imperial side above the junction of the Ockerson and C. D. Co. levees. The river has passed more than 120 000 sec-ft. at this gauge height and the curves on Fig. 5 show that if the rate of increase in flow is properly retarded, the river will pass at least 80 000 sec-ft. at this stage. Below the 16-mile point, the banks are not so high above low water and the river shows a disposition to widen and not cut so deep at high water, so that the banks near the Pescadero Cut-off now overflow at a discharge of 35 000 to 50 000 sec-ft. (See Fig. 6.)

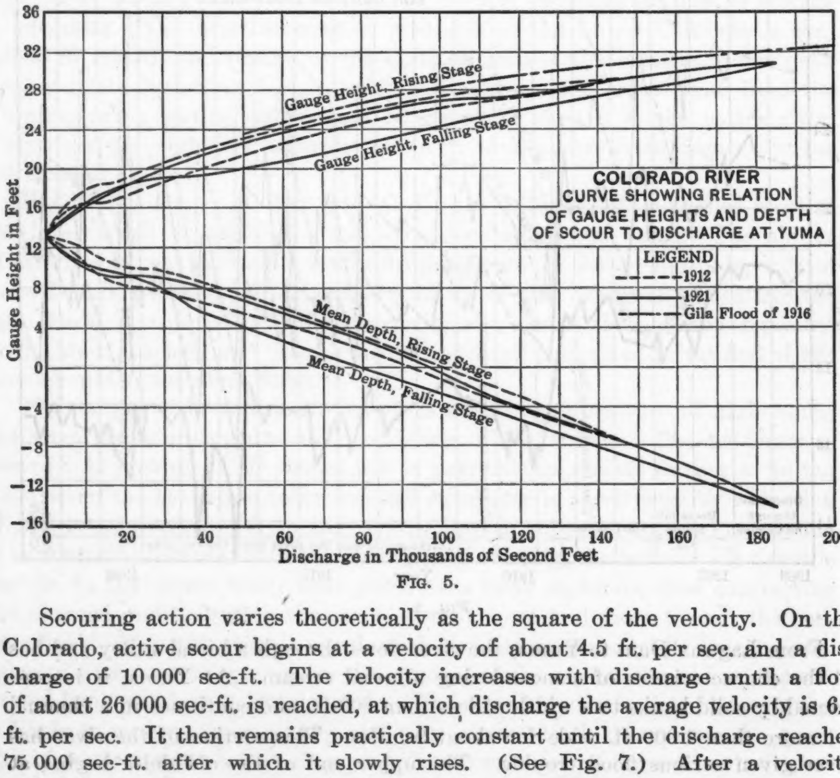

FIG. 5.

Scouring action varies theoretically as the square of the velocity. On the Colorado, active scour begins at a velocity of about 4.5 ft. per sec. and a discharge of 10 000 sec-ft. The velocity increases with discharge until a flow of about 26 000 sec-ft. is reached, at which discharge the average velocity is 6.2 ft. per sec. It then remains practically constant until the discharge reaches 75 000 sec-ft. after which it slowly rises. (See Fig. 7.) After a velocity of 6.2 ft. per sec. is reached, the increase in area of channel, due to scour and rise of water level, keeps pace with the increase in discharge. (See Figs. 5 and 8.) A rapid increase in discharge produces higher water levels than a slow increase, because there is not time for the scour to take place. This may be seen by examination of the discharge and gauge-height curves. (See Fig. 9.) Between 20 000 and 40 000 sec-ft. appears to be a critical stage on a rising river. The silt carried within that range of discharge is excessive (see Fig. 10) and

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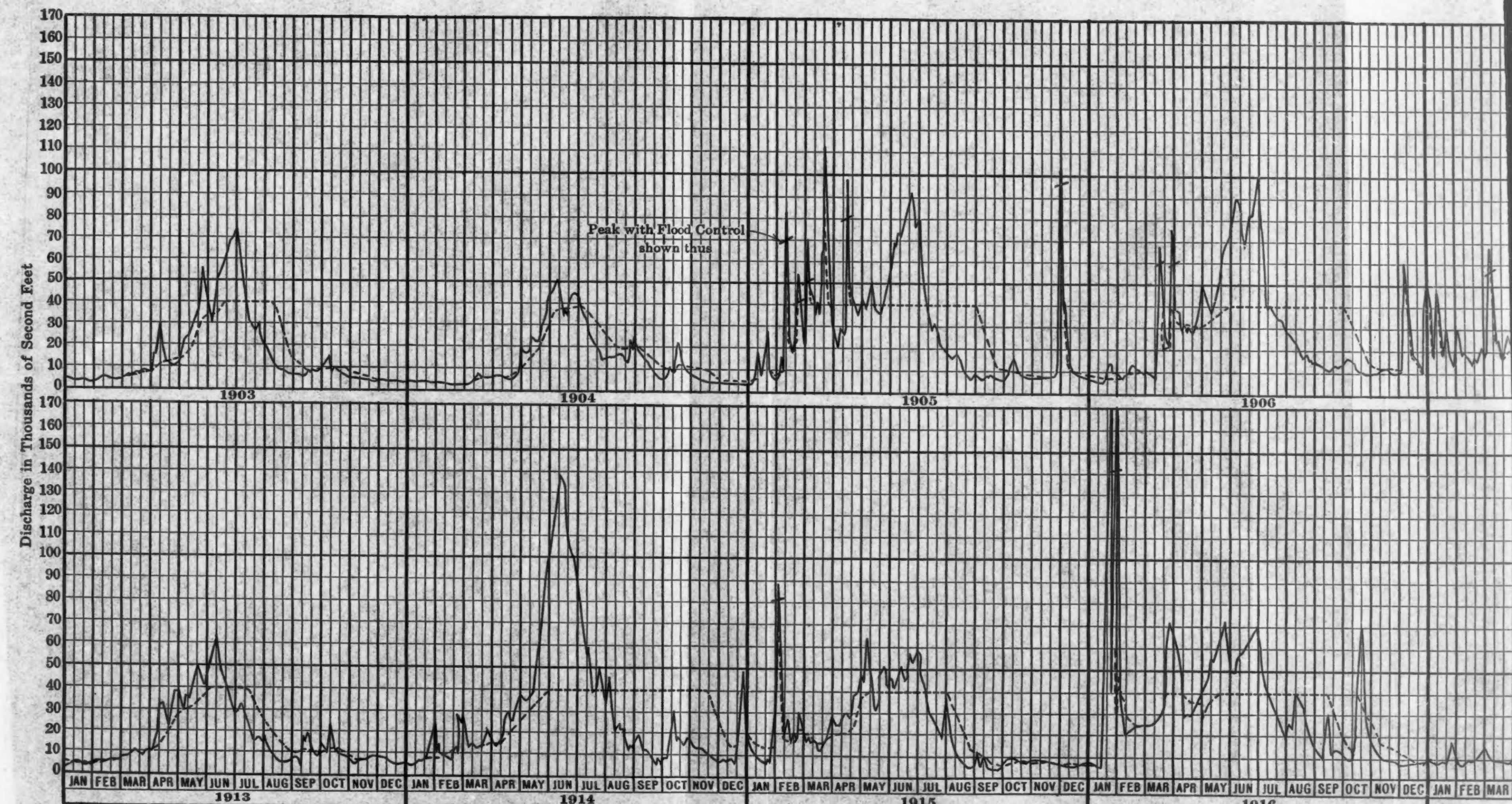

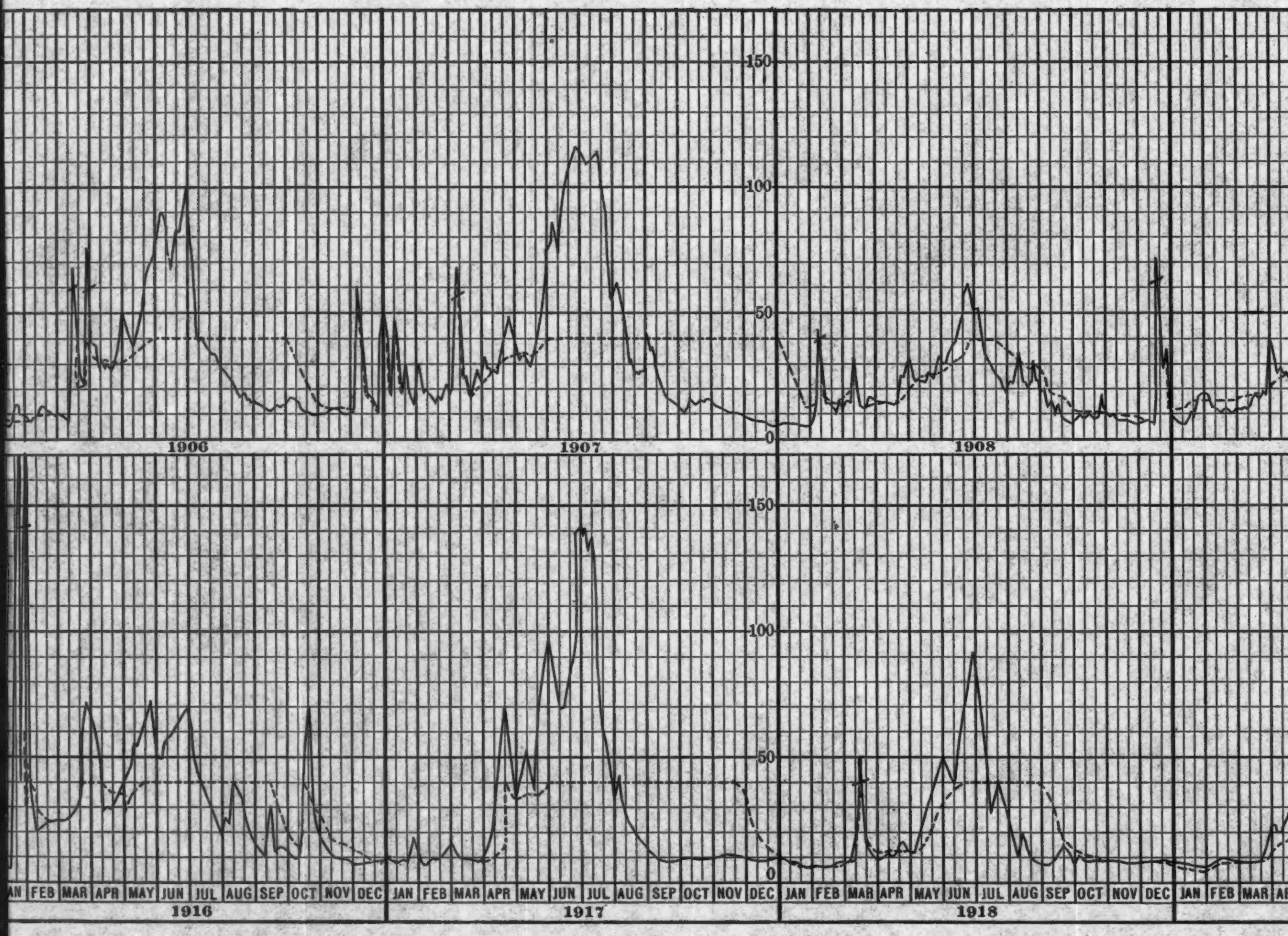

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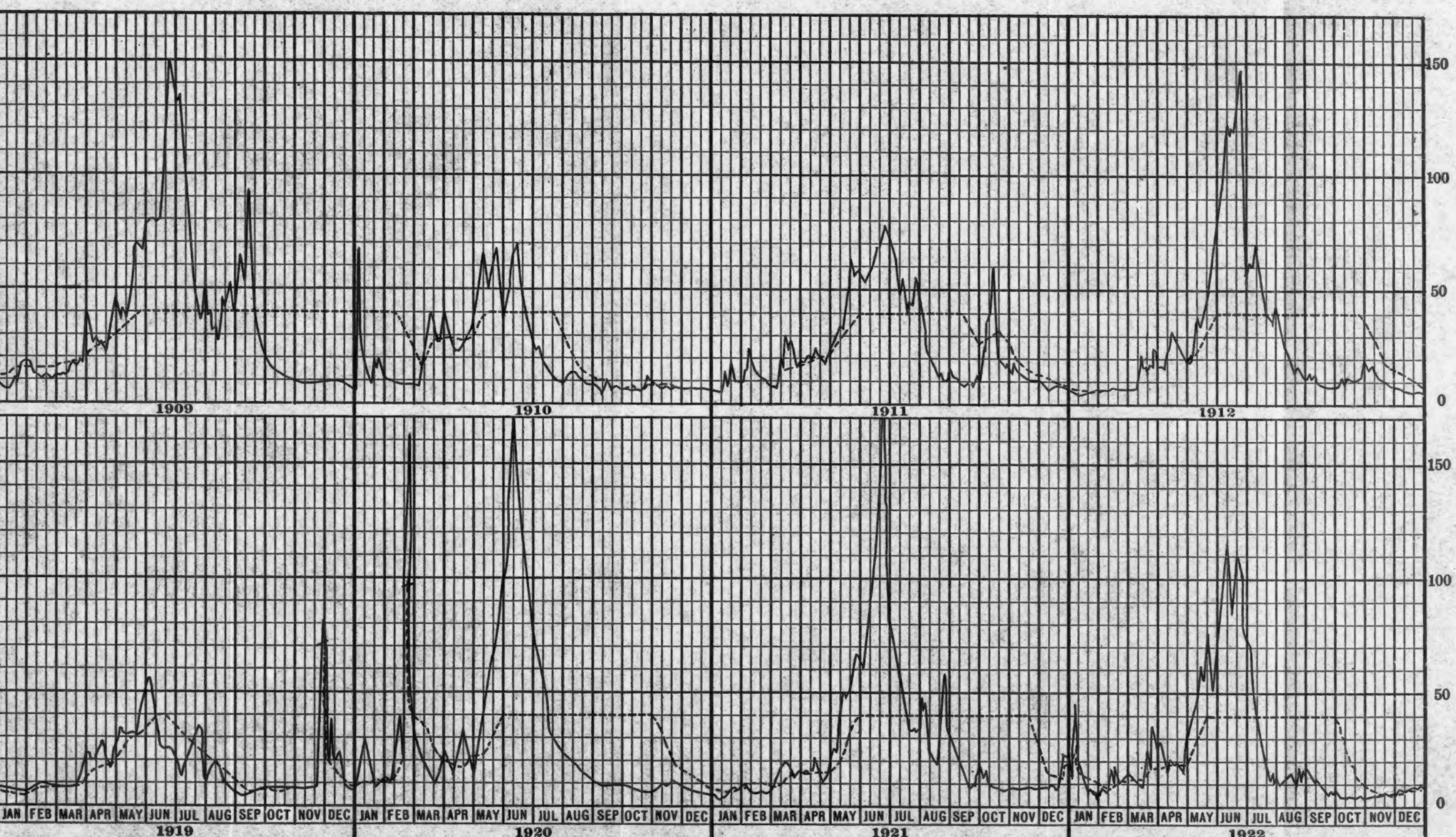
PLATE V.
PAPERS, AM. SOC.
AUGUST, 1924
KELLY ON
THE COLORADO RIVER



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NOTE

Increase in Diversions by

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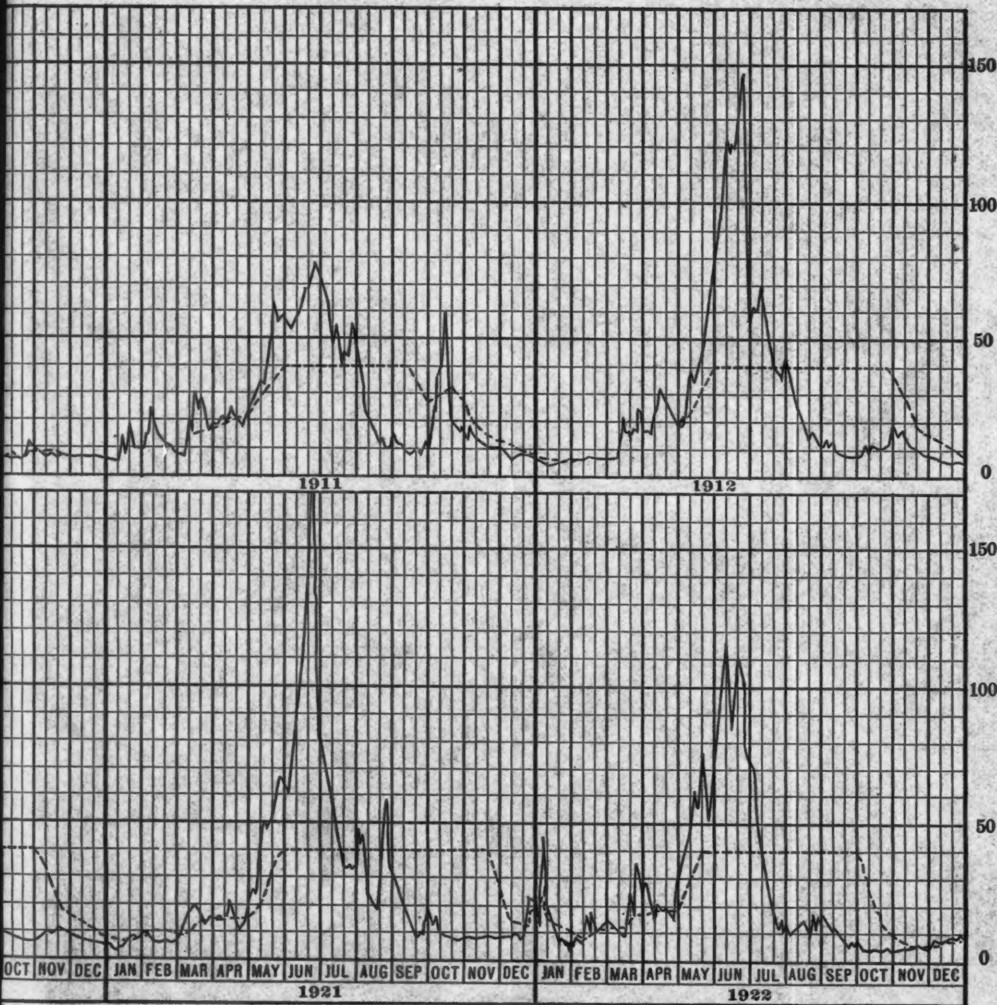

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PLATE V.
PAPERS, AM. SOC. C. E.
AUGUST, 1924
KELLY ON
THE COLORADO RIVER PROBLEM

ISCHARGE OF COLORADO RIVER
T YUMA AS RECORDED AND AS
IT WOULD BE WITH STORAGE
FOR FLOOD CONTROL ONLY

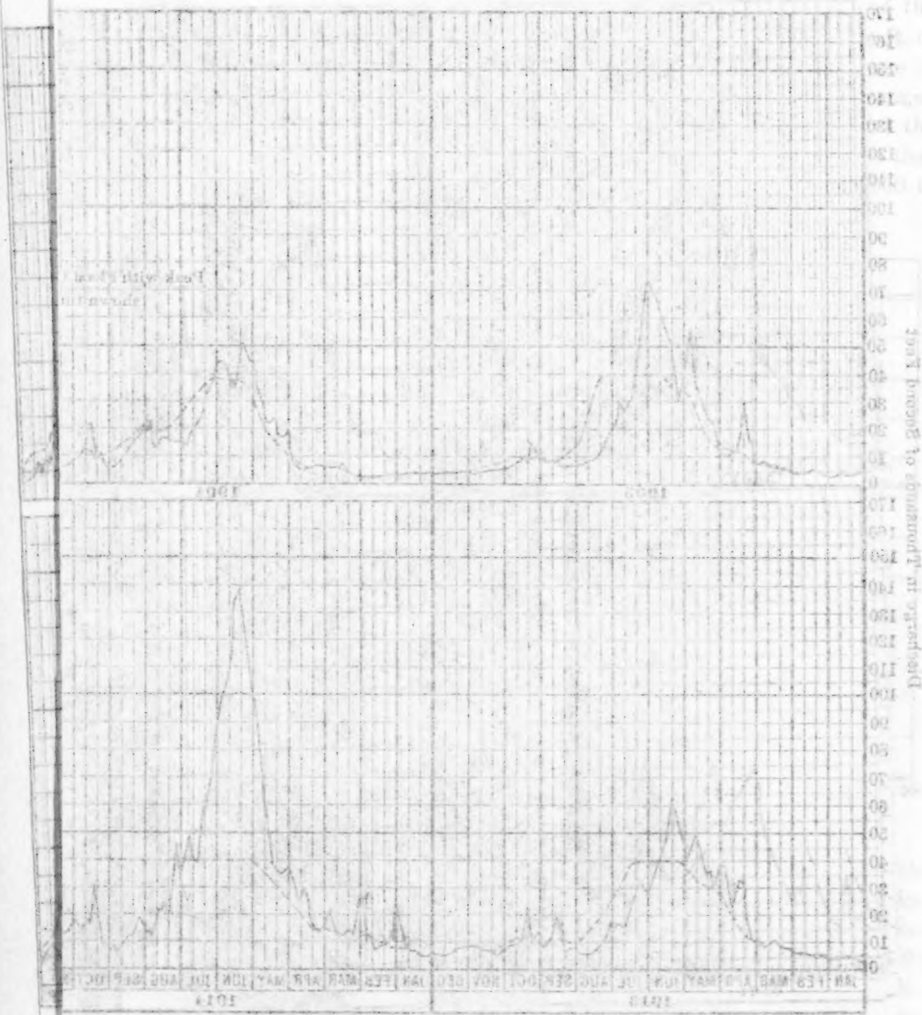
LEGEND

———— Discharges at Yuma as recorded

- - - - - - Discharges with Flood Control Reservoir of 10,000,000 Acre Feet Capacity and Regulation to 40,000 Sec. Ft. at Yuma

NOTE

Increase in Diversions by Upstream Development neglected



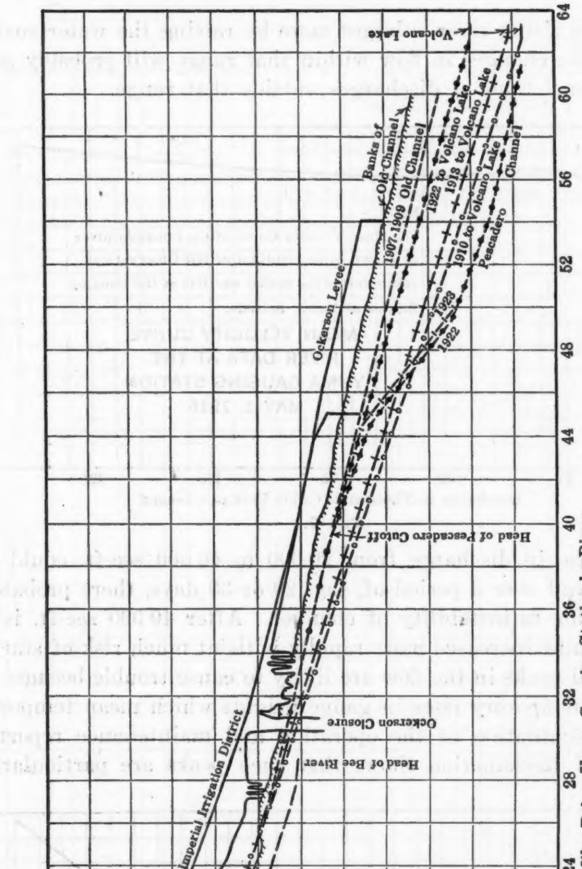

COLORADO RIVER
WATER SURFACE PROFILES
BELOW YUMA
April 12, 1907 28,000 Cu. Ft. per Sec.
June 1910 30 000
Dec. 19, 1912 5 000
July 7, 1913 32 000
May 1, 1922 30 000
1923
Elevation in Feet above Sea Level
Miles Below Yuma Gauging Station by River
Yuma Levee
California Development Company Levee
Present Levee of Imperial Irrigation District
Ockerson Levee
Imperial District Levee
Andrade Weir
Yuma Gauging Sta.
Head of Bee River
Ockerson Closure
Head of Pescadero Cutoff
Banks of Old Channel
1907-1909 Old Channel
1922 to Volcano Lake
1913 to Volcano Lake
1910 to Volcano Lake
Pescadero Channel
Volcano Lake

FIG. 6.

the channel on a rising river enlarges more by raising the water surface than by scour. Sudden changes in flow within that range will probably start more changes in channel than at discharges outside that range.

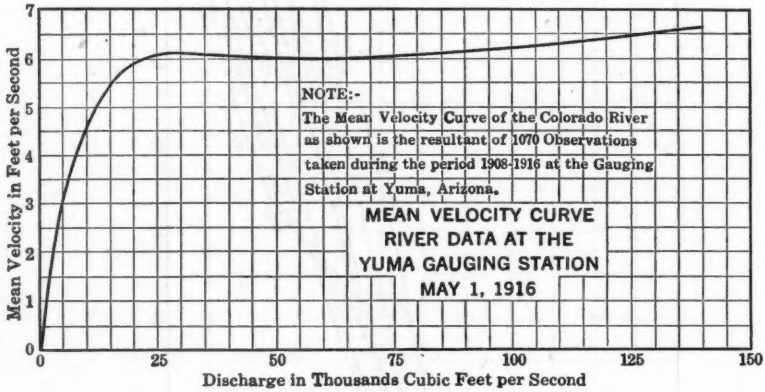


FIG. 7.

If the increase in discharge from 20 000 to 40 000 sec-ft. could be made gradual and extend over a period of, say, 20 or 30 days, there probably would be less trouble due to instability of channel. After 40 000 sec-ft. is reached, the discharge can be increased more rapidly without much risk of starting new meanders. Small peaks in the flow are likely to cause trouble because they are accompanied by temporary rises in gauge heights which mean temporary rises in velocity. Examination of the operation and maintenance reports of the U. S. Bureau of Reclamation shows that such peaks are particularly objec-

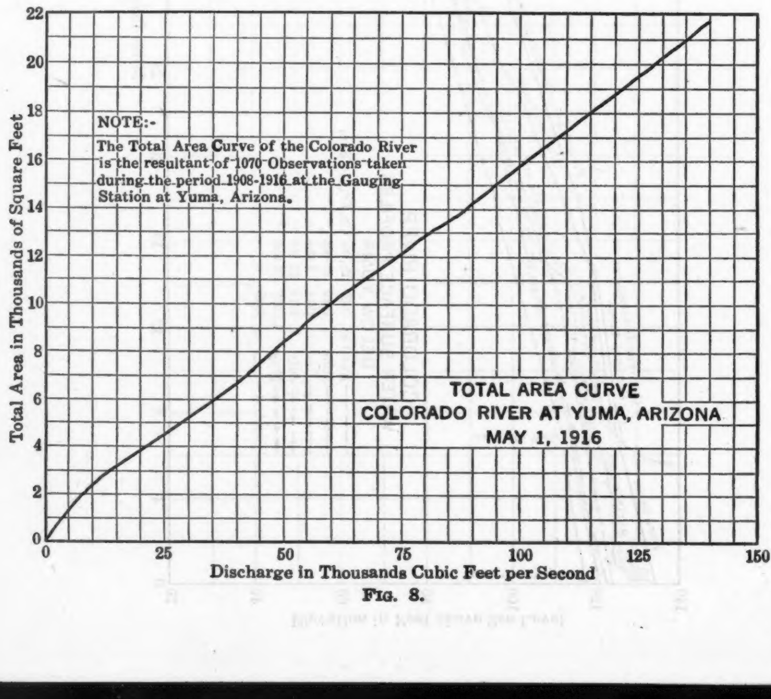


FIG. 8.

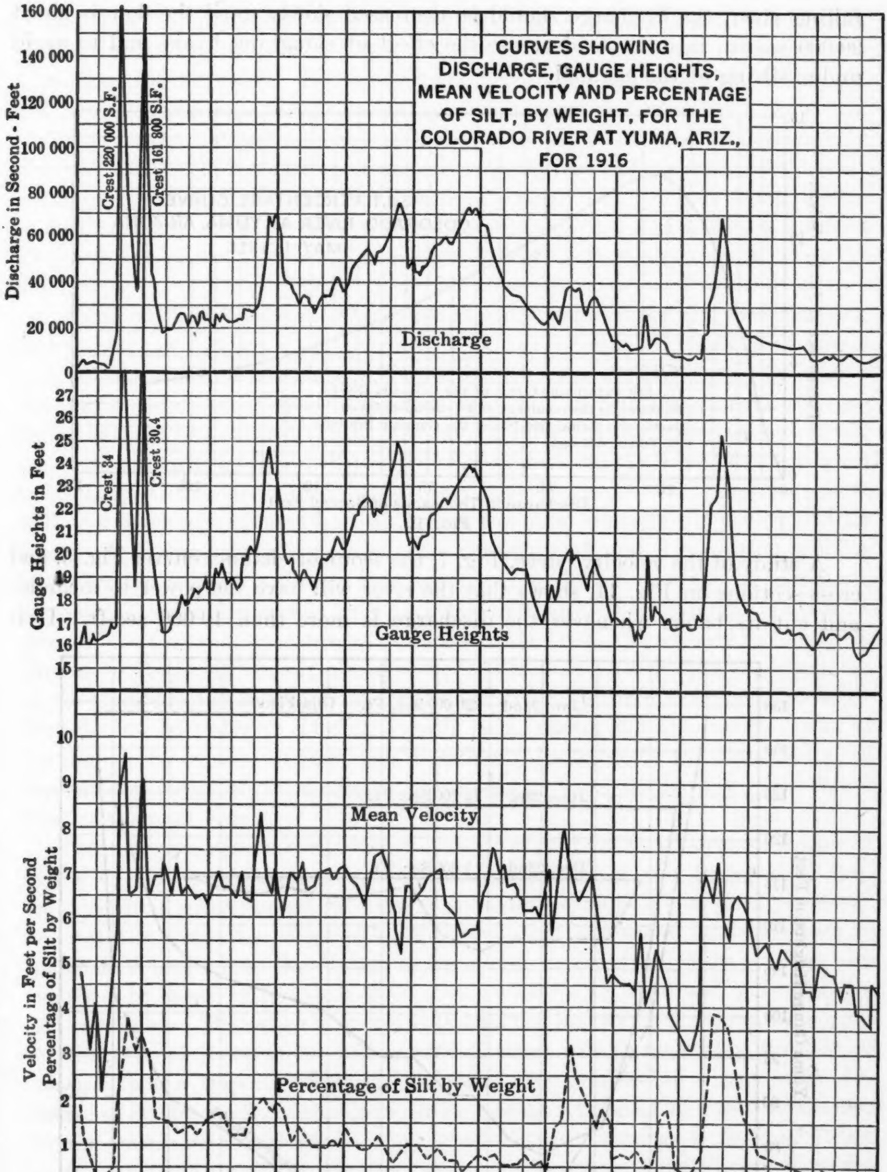

CURVES SHOWING DISCHARGE, GAUGE HEIGHTS, MEAN VELOCITY AND PERCENTAGE OF SILT, BY WEIGHT, FOR THE COLORADO RIVER AT YUMA, ARIZ., FOR 1916
Discharge in Second - Feet
160 000
140 000
120 000
100 000
80 000
60 000
40 000
20 000
0
Crest 220 000 S.F.
Crest 161 800 S.F.
Discharge
Gauge Heights in Feet
27
26
25
24
23
22
21
20
19
18
17
16
15
Crest 34
Crest 30.4
Gauge Heights
Velocity in Feet per Second
Percentage of Silt by Weight
10
9
8
7
6
5
4
3
2
1
0
Mean Velocity
Percentage of Silt by Weight
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.

FIG. 9.

tionable on a falling river. Such peaks on a falling river have been present nearly every time the undercutting of levees has gotten beyond control. On a falling river, the discharge should be decreased slowly until the flow is about 20 000 sec-ft., in order to decrease the effect of sloughing banks and to avoid undue silting of the channel.

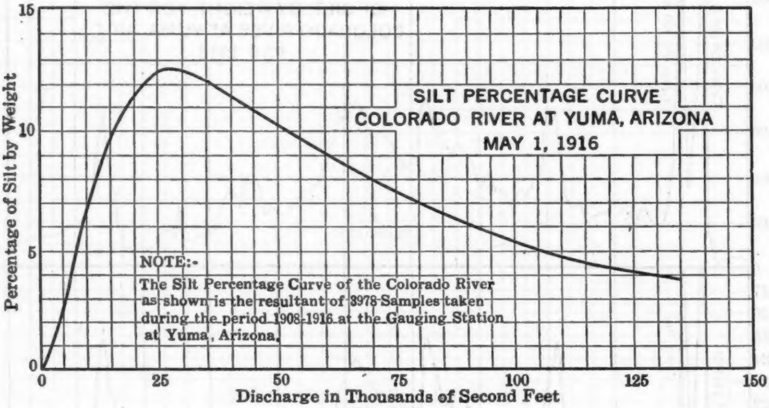


FIG. 10.

A study of the velocity curve, Fig. 7, the scour of channel curve, Fig. 5, and cross-sections on Fig. 11, shows that the river will have the power to meander and cut its banks whenever the discharge is more than 10 000 sec-ft. It is

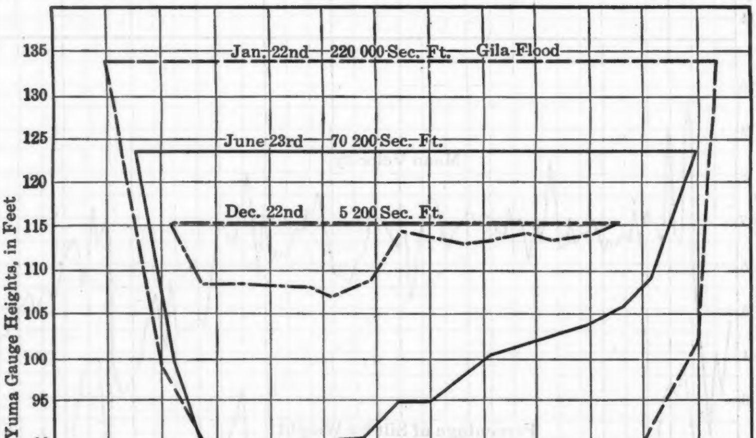


FIG. 11.

impossible to reduce the flow to that quantity, so storage of flood water will not eliminate the necessity for bank protection and levee maintenance. The records of past flood troubles show that the river has never gone beyond control sufficiently to damage property behind the levees, except when the discharge was more than 100 000 sec-ft. If the flow is reduced to a maximum of 80 000 sec-ft. and abrupt variations in discharge are eliminated, the river will be within its banks so that danger of property damage due to flooding will be reasonably removed for the Yuma and Imperial District Projects. It is doubtful whether reducing the maximum discharge below this point will produce much saving in cost of upkeep of flood-protection works on the Yuma and Imperial District Projects and there is small chance that it will reduce the cost of property damage.

Gila floods have occurred at various times between December 1 and March 1, and they come without much warning. As any storage dam on the Colorado will be 200 miles or more above Yuma, it will take at least three days for a change in discharge at the storage dam to be felt at Yuma and accurate regulation will not be possible. If the Colorado is regulated to a discharge of 40 000 sec-ft. as proposed by the U. S. Bureau of Reclamation, the flood waters will not be discharged before the middle of February in years like 1909. That means there will be a discharge of 40 000 sec-ft. in the Colorado through the period of Gila floods. It is impossible to predict what the effect of such a condition will be, but there is certainly a chance that damage from Gila floods may be increased, or that the irrigation supply may be interfered with, if an attempt is made to shut off the Colorado to conform to floods in the Gila, as proposed by the U. S. Bureau of Reclamation. It would seem wise to plan on reducing the discharge of the Colorado to the irrigation demand during the three months of probable Gila flood. In order to do this, it will be necessary for the outlet works at the dam to be capable of passing a maximum discharge in the Colorado of at least 75 000 sec-ft. so as to insure emptying the flood storage for the summer flood of the next year.

The most positive benefit that may be obtained by flood storage is that it will give insurance against permanent inundation of the Imperial Valley, provided it is great enough to permit releasing no more water than necessary to meet the irrigation demand over a period of low-water flow sufficient to enable any break into the Valley to be closed in the dry. Such a break can doubtless be closed in less than three weeks with the railroad and quarry equipment available. If storage is provided to hold the low-water flow for two months for the year of greatest run-off on record, there will be no reason for fear of disaster to the Imperial Valley. To make this practicable a maximum discharge of 80 000 sec-ft. in the Colorado is necessary in order to free flood storage for the succeeding flood season. If the maximum discharge is fixed at 75 000 sec-ft., the same result can be obtained by adding about 600 000 acre-ft. to the flood-storage capacity.

The engineers of the U. S. Bureau of Reclamation state that, considering diversions for irrigation since 1909, the flow of 1920 may be taken to represent the largest flow of record at Yuma. They give the storage required at Boulder Canyon for flood control as shown in Table 2.

TABLE 2.

Permissible discharge at Yuma, Ariz.	Required Storage, in Millions of Acre-Feet.		
	Excess flow at Yuma.	Stream storage.	Total.
40 000 sec-ft.	6.7	0.4	7.1
50 000 sec-ft.	5.5	0.4	5.9
60 000 sec-ft.	4.4	0.3	4.7
75 000 sec-ft.	3.2	0.2	3.4

The following conclusions seem to be justified:

1.—It is not feasible, with the quantity of water now passed by the Colorado, to relieve the Lower Basin from the expense of bank protection and levee maintenance by the construction of flood storage.

2.—Flood damage to property behind the levees will probably be avoided if the maximum discharge of the Colorado is reduced to 80 000 sec-ft., and there will be little, if any, reduction in cost of bank protection and maintenance by a greater reduction in discharge.

3.—It may be desirable, until the Gila is controlled, to reduce the flow in the Colorado during the months of December, January, and February, to that required for irrigation. This means that discharge facilities with a capacity of at least 75 000 sec-ft. should be provided.

4.—It is desirable to allay the fear of permanent inundation of the Imperial Valley by making it possible to stop completely the flow below Rockwood Heading for a period sufficiently long to permit remedial works to be carried out in the dry. This can be done by using the storage provided for flood control, if flood waters are discharged at a rate of 75 000 sec-ft. and an extra 600 000 acre-ft. of storage capacity is provided.

5.—Storage for flood control is justified to a quantity that will reduce the maximum discharge for any year of record to 75 000 sec-ft. Storage in excess of that quantity may be deferred until provided as an incident to other development on the river.

6.—The conditions mentioned will be met by 4 000 000 acre-ft. of flood storage. Additional storage to allow for silting of reservoirs may be advisable. The quantity of such storage necessary will depend on the shape, location, and method of operation of the storage reservoir and should be determined after the reservoir site is chosen.

7.—The reservoir should be designed so that when the summer flood starts, the discharge can be increased gradually without intermediate peaks. Insofar as the river discharge permits, the maximum discharge of 75 000 sec-ft. should be reached by the time the June peak arrives. After the June peak has passed and the inflow to the reservoir has diminished to less than 75 000 sec-ft., the discharge should be gradually reduced, avoiding intermediate peaks until the discharge is less than 20 000 sec-ft., in order to avoid excessive sloughing and undercutting of banks and excessive silting of the channel.

8.—For flood protection of the lower basin, the reservoir should be located as far down stream as practicable.

All-American Canal

The "Swing-Johnson" Bill provides that the United States Government finance and construct the so-called All-American Canal to serve the Imperial Valley and other lands in California, reimbursement to be sought from the lands benefited. The project is one that has been under consideration in one form or another for some time. It was reported on in July, 1919, by the "All-American Canal Board" and the report is printed in a public document entitled "The All-American Canal".

On October 23, 1918, an agreement was entered into between the Secretary of the Interior and the Imperial Irrigation District to provide for extension of Imperial Canal to Laguna Dam and committing the Imperial Irrigation District to the construction of an All-American Canal. This agreement was ratified by the voters of the Imperial Irrigation District on January 21, 1919, prior to the completion of the report by the All-American Canal Board, by a vote of 2 535 in favor and 922 against. Payments have been made by the Imperial District in accordance with the agreement so that the agreement is still in force but, on account of financial difficulties and the possibility that the United States might undertake the project, no construction work has been done.

Analysis of the All-American Canal project as proposed in the "Swing-Johnson" Bill shows that it may be considered as three separate propositions which have different degrees of urgency, as follows:

(1).—Connection with Laguna Dam so that water for lands on the west side of the river may be diverted at that point instead of at the present Rockwood Heading.

(2).—Serving lands in the Imperial Irrigation District through a canal entirely on the American side of the International Boundary in order to avoid complications that have arisen in the past over the operation and maintenance of a canal in Mexico.

(3).—Serving lands on the East Side Mesa and in Coachella Valley which are too high to be served by the present canal system.

The proposition to connect with Laguna Dam has been under consideration for some time. It would overcome the difficulties and dangers incident to diversion at the Rockwood Heading and would materially reduce the cost of maintaining the main Imperial Canal by keeping a considerable part of the silt out of it. It would also incidentally reduce the construction costs against the Yuma Project and furnish power for its pumping area. This part of the project will cost about $6.50 per acre for the Imperial District, and the cost will be practically the same whether the connection is made to the present Imperial Canal or to the proposed All-American Canal. The Imperial District is on record as ready to pay its share of the undertaking and there is no doubt that the project should be carried out. The only question at issue is whether the Federal Government shall finance the proposition and that is a matter of policy to be determined by Congress.

The "All-American Canal" proposition is the result of international complications incident to operating the main Imperial Canal through Mexico. Physically, the present canal is in excellent condition. It follows the natural line for a canal and if connected to Laguna Dam and relieved of Mexican

troubles would be more satisfactory than the proposed All-American Canal which must cross drifting sand hills and difficult terrain for about 15 miles in cuts more than 100 ft. deep. The cost of the "All-American Canal" to the Imperial District, if undertaken alone, would be about $40 per acre. If undertaken in conjunction with all other lands possible of irrigation therefrom, the cost to the Imperial District will be about $30 per acre. These costs are additional to the $6.50 per acre for connecting to Laguna Dam. The question of whether the advantages of getting out of Mexico justify the cost is one for the Imperial District to decide. It did decide in 1919, but since that time the estimate of cost has been increased and the Mexican Government has been recognized, so that the prospect of relief by treaty is improved. If the Imperial District is ready to stand by its agreement to pay for this project, as it appears to be, the only question at issue is, shall the United States finance and build the project? Decision on this question is more or less dependent on the decision on the third proposition.

If the United States decides to undertake the irrigation of the lands too high to be reached by present canals, there will be advantages in combining with the Imperial District. The cost of an "All-American Canal" for the 270 000 acres that can be irrigated outside the Imperial District will be about $62.50 per acre, if undertaken alone, and $31 per acre, if undertaken in conjunction with the Imperial District. To these amounts should be added $6.50 per acre for connection with Laguna Dam and the cost of distribution systems. There is no information available to show the cost of a high canal through Mexico on a more favorable route, nor the cost of serving this land by pumping from the present canal system.

TABLE 3.—COST OF CONNECTING THE IMPERIAL CANAL (OR ALL-AMERICAN CANAL) WITH THE LAGUNA DAM.

	Including power.	Power charged separately.
Case I.—All-American Canal as Projected. (Capacity 9 000 sec-ft. + 1 600 sec-ft.):		
Yuma Project (114 000 acres)	$13.00	$33.00 pump lands 0.00 non-pump lands
Imperial Irrigation District (515 000 acres)	8.00	6.50
New land under All-American Canal (300 000 acres)	8.00	6.50
Case II.—Mexican Lands now Under Imperial Canal Added to All-American Canal Area. (Capacity 11 500 sec-ft. + 1 600 sec-ft.):		
Yuma Project (114 000 acres)	13.00	33.00 pump lands 0.00 non-pump lands
Imperial Irrigation District (515 000 acres)	6.50	5.75
New lands under All-American Canal (300 000 acres)	6.50	5.75
Mexican lands under Imperial Canal (255 000 acres)	6.50	5.75
Case III.—Connection Made with Imperial Canal and Water Distributed Through That System (8 500 sec-ft. + 1 600 sec-ft.):		
Yuma Project (114 000 acres)	13.00	33.00 pump lands 0.00 non-pump lands
Imperial Irrigation District (515 000 acres)	8.00	6.50
Mexican lands under Imperial Canal (255 000 acres)	8.00	6.50

Tables 3 and 4 were prepared in order to analyze the costs of various parts of the project. They are based on unit costs of the "All-American Canal Board"

and acreages given in the "Fall-Davis" report. It will be noted that if the Imperial District and Mexican high lands participate in the "All-American Canal", the cost of putting water on the East Mesa lands will be about $81 per acre, and on Coachella Valley lands about $120 per acre. If the Imperial District and Mexican high lands do not participate in the "All-American Canal", these costs will be raised to $113 and $153, respectively. On the East Side Mesa, 148 000 acres of the 160 000 irrigable are public lands. In the Coachella Valley and Dos Palmas combined, of a total of 77 000 acres irrigable, 4 500 acres are public lands and 11 400 acres are Indian lands. If the East Side Mesa and Coachella Valley lands are combined so as to average the costs, the figures become $90 per acre and $118 per acre, respectively, depending on whether or not the Imperial District participates in the "All-American Canal". The data available are not sufficient to determine how much cost is justifiable to bring the 270 000 acres of lands outside Imperial District under irrigation.

TABLE 4.—Cost of All-American Canal.

(Including Cost of Connection with Laguna Dam. Yuma Project Charged for Its Interest in the Improvement.)

	Distribution system and power.	All-American Canal.	Total per acre.
Case I.—Cost if Built for New Lands Only (Excluding West Side Area) (Capacity 3 000 sec-ft.):			
(*A*). Cost of All-American Canal, not including cost of distribution systems and power installation: $18 321 000 for 265 600 acres at $69.00 per acre.			
(*B*). Cost of All-American Canal with distribution systems and power installation included:			
"A" Line Canal south of Iris (98 000 acres)........	$44.70	$69.00	$113.70
"A" Line Canal north of Iris (77 100 acres)........	84.00	69.00	153.00
South Side Gravity:			
(Mexico 22 000 acres; United States 8 800 acres)..	20.00	69.00	89.00
"E" Line (15 700 acres)........	23.50	69.00	92.50
"D" Line (Pump) (44 000 acres)........	30.60	69.00	99.60
Case II.—Cost if Built for Imperial Irrigation District and New Lands Listed in Senate Document No. 142* (Capacity 9 000 sec-ft.):			
(*A*). Cost of All-American Canal, not including cost of distribution system and power installations: $29 793 000 for 813 600 acres at $36.60 per acre; $29 793 000 for 736 500 acres at $40.40 per acre. (North of Iris excluded.)			
(*B*). Cost of All-American Canal per acre with cost of distribution systems and power installations included:			
Imperial Irrigation District (515 000 acres).........	4.00	36.60	40.90
"A" Line Canal south of Iris (98 000 acres)........	44.70	36.60	81.30
"A" Line Canal north of Iris (77 100 acres)........	84.00	36.60	120.60
South Side Gravity:			
(Mexico 22 000 acres; United States 8 800 acres.)...	20.00	36.60	56.60
"E" Line (15 700 acres)........	23.50	36.60	60.10
"D" Line (Pump) (44 000 acres)........	30.60	36.60	67.20
"B" Line (Gravity and Pump) (33 000 acres)......	56.00	36.60	92.60
Case III.—Cost if Built for Imperial Irrigation District Alone (Capacity, 6 000 sec-ft.):			
Cost of All-American Canal, without power installation or new distribution systems, but prepared for later enlargement to take care of new lands: $24 504 057†, or 515 000 acres at $47.60 per acre.			

* 67th Congress, 2d Sess.

† All American Canal Rept., p. 96.

WATER SUPPLY, STORAGE REQUIRED, AND IRRIGABLE AREAS

The following is extracted from a report recently made to the Secretary of the Interior by Herman Stabler, M. Am. Soc. C. E., Chief of the Land Classification Branch of the U. S. Geological Survey:

"WATER SUPPLY OF COLORADO RIVER

"The run-off of Colorado River available for future development above the diversion for the Yuma project at Laguna Dam and for all development below this point can be expressed in terms of discharge at Laguna Dam, obtained by subtracting from the run-off of the Colorado at Yuma (*a*) the run-off of the Gila at Yuma and (*b*) depletion by reason of irrigation use and diversions from the basin above, and adding to the result thus obtained the diversions for the Yuma Project.

"In this study there have been used (*a*) records of discharge of the Colorado at Yuma since 1902; (*b*) rough estimates of discharge of the Gila at Yuma since 1903; (*c*) records of diversions for the Yuma Project since 1910; and (*d*) estimates of depletion since 1899, all furnished by the Bureau of Reclamation. In addition, gauge heights of Colorado River at Yuma from 1878 to 1922 have been used to extend the record of discharge back to 1878, and the estimates of depletion have also been extended on the assumption that irrigation in the basin began about 1850. Records of run-off of Salt River since 1889 and estimates of inflow to Great Salt Lake under the assumption of present-day development estimated from the record of Salt Lake levels have been studied as comparates, but have not been used in estimates of run-off.

"Two methods were used to develop a relation between discharge and stage at Yuma, effort being made in both to establish a method by which allowance for shifting channel conditions could be made from the record of gauge heights alone. Both methods give similar results in years of moderate flow. One method gives results rather high for years of low flow, while the other appears to give results slightly low for years of low flow. The average results by the two methods are presented as probably being more reliable than either, though apparently indicating discharges too great for years of low flow. For the period 1902-1922, in which comparison with discharge records is possible, the annual results obtained are less than the recorded discharge by as much as 10% in three years (1907, 1914, and 1920), the maximum being 20% for the year 1907. Considered as 5-year progressive means, the period 1906-1910 is found to be lower than the recorded discharge by 5%, while the period 1902-1907 is higher than the recorded discharge by 7.6%, these representing the maximum digressions. These results check the accuracy of the method so far as it can be tested directly and indicate that it is fairly reliable as an indicator of mean discharge for periods of 5 years or more.

"The discharge of the Colorado as estimated for the period 1878-1922 at the diversion for the Yuma Project is given in following table [Table 5], in millions of acre-feet.

"The following table [Table 6], shows the mean discharge for various periods for Colorado River, Great Salt Lake, and Salt River, based on mean discharge for the period 1902-1922, the period of discharge records on Colorado River, all on the assumption that conditions of development were the same as in 1922 throughout the periods considered.

"The comparison here afforded tends to increase materially the confidence with which the estimated discharge prior to 1902 may be used.

"EFFECT OF STORAGE ON WATER SUPPLY

"Studies of water supply indicate that under conditions of development as in 1922, the water supply likely to reach Laguna Dam varies from about 6 000 000 to about 27 000 000 acre-ft. a year, and it is not unlikely that these

limits may be exceeded in both directions. The mean flow (1922 conditions and equivalent flow at Laguna) for a period of 45 years, 1878-1922, is between 13 000 000 and 14 000 000 acre-ft. For periods of as much as or more than 20 years, however, the flow may be less than 11 000 000 or more than 16 000 000 acre-ft.

"TABLE 5.—Estimated Discharge of Colorado River at Laguna with 1922 Conditions of Development.

Year.	Discharge.	Year.	Discharge.	Year.	Discharge.
1878	13.0	1893	11.7	1908	11.7
1879	8.7	1894	8.8	1909	24.6
1880	13.6	1895	9.4	1910	13.3
1881	11.2	1896	7.7	1911	16.8
1882	8.6	1897	10.2	1912	17.5
1883	14.0	1898	7.7	1913	11.5
1884	27.2	1899	14.0	1914	20.1
1885	15.3	1900	10.1	1915	12.6
1886	11.5	1901	14.2	1916	18.7
1887	9.1	1902	6.0	1917	19.6
1888	8.3	1903	10.3	1918	13.0
1889	9.4	1904	9.0	1919	10.3
1890	14.5	1905	14.9	1920	21.1
1891	12.9	1906	16.6	1921	19.5
1892	14.4	1907	23.9	1922	16.9
Mean	....	....	....	....	13.6

"Irrigation and power requirements both would be best served if the annual flow were made substantially uniform. The following tables [Tables 7, 8, 9, and 10], indicate what may be accomplished in this regard with specified amounts of storage capacity under the assumption that the storage reservoir or reservoirs would be full or, if of sufficient capacity, would contain 10 000 000 acre-ft. at the beginning of the 45-year period considered. The tables show the amount of water in storage each year, evaporation throughout being assumed to be 3% of the amount of water in storage. An empty reservoir and shortage in available water supply is indicated by parentheses and minus sign.

"TABLE 6.—Mean Discharge in Percentage of Mean Discharge for Period 1902-1922, with 1922 Conditions of Development.

Period.	Colorado River at Laguna.	Great Salt Lake inflow.	Salt River at Roosevelt.
1902-1922	100	100	100
1899-1922	98	97	92
1889-1922	89	90	90
1878-1922	87	89	..
1851-1922	..	92	..
1889-1901	72	78	72
1878-1901	76	78	..
1886-1904	67	75	..
1889-1904	68	75	64

"In the foregoing summary the percentage of maximum shortage is based on the stated amount of annual demand. As irrigation development proceeds, the annual flow at the storage reservoirs would be decreased by more than 1 000 000 acre-ft. when the area of lands estimated to be irrigable in the immediate future

"TABLE 7.—SHOWING SURPLUS OR DEFICIENCY OF WATER SUPPLY WITH REGULATION IN A RESERVOIR OF 5 000 000 ACRE-FT. CAPACITY FOR SPECIFIED ANNUAL DEMAND.
(Millions of Acre-Feet.)

Years.	Estimated flow at Laguna.	ANNUAL DEMAND: 10.	11.
1877	....	5.0	5.0
1878	13.0	5.0	5.0
1879	8.7	3.5	2.5
1880	13.6	5.0	5.0
1881	11.2	5.0	5.0
1882	8.6	3.5	2.5
1883	14.0	5.0	5.0
1884	27.2	5.0	5.0
1885	15.3	5.0	5.0
1886	11.5	5.0	5.0
1887	9.1	3.9	2.9
1888	8.3	2.1	0.1
1889	9.4	1.5	(—1.5)
1890	14.5	5.0	3.5
1891	12.9	5 0	5.0
1892	14.4	5.0	5.0
1893	11.7	5.0	5.0
1894	8.8	3.6	2.6
1895	9.4	2.9	0.9
1896	7.7	0.5	(—2.4)
1897	10.2	0.7	(—0.8)
1898	7.7	(—0.7)	(—3.3)
1899	14.0	4.0	3.0
1900	10.1	4.0	2.0
1901	14.2	5.0	5.0
1902	6.0	0.8	(—0.2)
1903	10.3	1.1	(—0.7)
1904	9.0	0.0	(—2.0)
1905	14.9	4.9	3.9
1906	16.6	5.0	5.0
1907	13.9	5.0	5.0
1908	11.7	5.0	5.0
1909	24.6	5.0	5.0
1910	13.3	5.0	5.0
1911	16.8	5.0	5.0
1912	17.5	5.0	5.0
1913	11.5	5.0	5.0
1914	20.1	5.0	5.0
1915	12.6	5.0	5.0
1916	18.7	5.0	5.0
1917	19.6	5.0	5.0
1918	13.0	5.0	5.0
1919	10.3	5.0	4.1
1920	21.1	5.0	5.0
1921	19.5	5.0	5.0
1922	16.9	5.0	5.0

"TABLE 8.—SHOWING SURPLUS OR DEFICIENCY OF WATER SUPPLY WITH REGULATION IN A RESERVOIR OF 10 000 000 ACRE-FT. CAPACITY FOR SPECIFIED ANNUAL DEMAND.
(Millions of Acre-Feet.)

Years.	Estimated flow at Laguna.	ANNUAL DEMAND:					
		10.	11.	11.5.	12.	12.5.	13.
1877	....	10.0	10.0	10.0	10.0	10.0	10.0
1878	13.0	10.0	10.0	10.0	10.0	10.0	9.7
1879	8.7	8.7	7.7	6.9	6.4	5.9	5.1
1880	13.6	10.0	10.0	8.8	7.8	6.8	5.5
1881	11.2	10.0	9.9	8.2	6.8	5.3	3.5
1882	8.6	8.6	7.2	5.1	3.2	1.2	(—1.0)
1883	14.0	10.0	10.0	7.4	5.1	2.7	1.0
1884	27.2	10.0	10.0	10.0	10.0	10.0	10.0
1885	15.3	10.0	10.0	10.0	10.0	10.0	10.0
1886	11.5	10.0	10.0	9.7	9.2	8.7	8.2
1887	9.1	9.1	7.8	7.0	6.0	5.0	4.1
1888	8.3	7.1	4.8	3.6	2.1	0.6	(—0.7)
1889	9.4	6.3	3.0	1.4	(—0.6)	(—2.1)	(—3.6)
1890	14.5	10.0	6.4	4.4	2.5	2.0	1.5
1891	12.9	10.0	8.1	5.7	3.3	2.3	1.4
1892	14.4	10.0	10.0	8.4	5.6	4.1	2.8
1893	11.7	10.0	10.0	8.4	5.1	3.2	1.4
1894	8.8	8.8	7.5	5.4	1.7	(—0.6)	(—2.8)
1895	9.4	7.9	5.7	3.1	(—1.0)	(—3.1)	(—3.6)
1896	7.7	5.4	2.2	(—0.8)	(—4.3)	(—4.8)	(—5.3)
1897	10.2	5.4	1.3	(—1.3)	(—1.8)	(—2.3)	(—2.8)
1898	7.7	3.0	(—2.0)	(—3.8)	(—4.3)	(—4.8)	(—5.3)
1899	14.0	6.9	3.0	2.5	2.0	1.5	1.0
1900	10.1	6.8	2.0	1.5	0.0	(—0.9)	(—1.9)
1901	14.2	10.0	5.1	4.2	2.2	1.7	1.2
1902	6.0	6.0	0.0	(—0.6)	(—3.9)	(—4.8)	(—5.8)
1903	10.3	6.1	(—0.7)	(—1.2)	(—1.7)	(—2.2)	(—2.7)
1904	9.0	4.9	(—2.0)	(—2.5)	(—3.0)	(—3.5)	(—4.0)
1905	14.9	9.7	3.9	3.4	2.9	2.4	1.9
1906	16.6	10.0	9.4	8.4	7.4	6.4	5.4
1907	13.9	10.0	10.0	10.0	9.1	7.6	6.1
1908	11.7	10.0	10.0	9.9	8.5	6.6	4.6
1909	24.6	10.0	10.0	10.0	10.0	10.0	10.0
1910	13.3	10.0	10.0	10.0	10.0	10.0	10.0
1911	16.8	10.0	10.0	10.0	10.0	10.0	10.0
1912	17.5	10.0	10.0	10.0	10.0	10.0	10.0
1913	11.5	10.0	10.0	9.7	9.2	8.7	8.2
1914	20.1	10.0	10.0	10.0	10.0	10.0	10.0
1915	12.6	10.0	10.0	10.0	10.0	10.8	9.3
1916	18.7	10.0	10.0	10.0	10.0	10.0	10.0
1917	19.6	10.0	10.0	10.0	10.0	10.0	10.0
1918	18.0	10.0	10.0	10.0	10.0	10.0	9.7
1919	10.3	10.0	9.0	8.5	8.0	7.5	6.7
1920	21.1	10.0	10.0	10.0	10.0	10.0	10.0
1921	19.5	10.0	10.0	10.0	10.0	10.0	10.0
1922	16.9	10.0	10.0	10.0	10.0	10.0	10.0

"TABLE 9.—Showing Surplus or Deficiency in Water Supply with Regulation in a Reservoir of 20 000 000 Acre-Ft. Capacity for Specified Annual Demand.
(Millions of Acre-Feet.)

Years.	Estimated flow at Laguna.	Annual Demand:			
		11.	12.	13.	14.
1877	....	10.0	10.0	10.0	10.0
1878	13.0	11.7	10.7	9.7	8.7
1879	8.7	9.1	7.1	5.1	3.1
1880	13.6	11.4	7.8	5.5	2.6
1881	11.2	11.3	6.8	3.5	(—0.3)
1882	8.6	8.6	3.2	(—1.0)	(—5.4)
1883	14.0	11.3	5.1	1.0	(—0.0)
1884	27.2	20.0	20.0	15.2	13.2
1885	15.3	20.0	20.0	17.1	14.1
1886	11.5	19.9	18.9	15.1	11.2
1887	9.1	17.4	15.4	10.8	6.0
1888	8.3	14.2	11.2	5.8	0.1
1889	9.4	12.2	8.3	2.0	(—4.5)
1890	14.5	15.3	10.6	3.4	0.5
1891	12.9	16.7	10.7	3.2	(—0.6)
1892	14.4	19.6	12.8	4.5	0.4
1893	11.7	19.7	12.1	3.1	(—1.9)
1894	8.8	16.9	8.5	(—1.2)	(—5.2)
1895	9.4	14.8	5.6	(—3.6)	(—4.6)
1896	7.7	11.1	1.1	(—5.3)	(—6.3)
1897	10.2	10.0	(—0.7)	(—2.8)	(—8.8)
1898	7.7	6.4	(—4.3)	(—5.3)	(—6.3)
1899	14.0	9.2	2.0	1.0	0.0
1900	10.1	8.0	0.0	(—1.9)	(—3.9)
1901	14.2	11.0	2.2	1.2	0.2
1902	6.0	5.7	(—3.9)	(—5.8)	(—7.8)
1903	10.3	4.8	(—1.7)	(—2.7)	(—3.7)
1904	9.0	2.6	(—3.0)	(—4.0)	(—5.0)
1905	14.9	6.4	2.9	1.9	0.9
1906	16.6	11.8	7.4	5.4	3.5
1907	13.9	14.3	9.1	6.1	3.3
1908	11.7	14.5	8.5	4.6	0.9
1909	24.6	20.0	20.0	16.1	11.5
1910	13.3	20.0	20.0	15.9	10.5
1911	16.8	20.0	20.0	19.2	13.0
1912	17.5	20.0	20.0	20.0	16.1
1913	11.5	19.9	18.9	17.9	13.1
1914	20.1	20.0	20.0	20.0	18.7
1915	12.6	20.0	20.0	19.0	16.7
1916	18.7	20.0	20.0	20.0	20.0
1917	19.6	20.0	20.0	20.0	20.0
1918	13.0	20.0	20.0	19.4	18.4
1919	10.3	18.7	17.7	16.3	14.1
1920	21.1	20.0	20.0	20.0	20.0
1921	19.5	20.0	20.0	20.0	20.0
1922	16.9	20.0	20.0	20.0	20.0

"TABLE 10.—Showing Surplus or Deficiency in Water Supply with Regulation in a Reservoir of 30 000 000 Acre-Ft. Capacity for Specified Annual Demand.

(Millions of Acre-Feet.)

Years.	Estimated flow at Laguna.	Annual Demand.		
		12	13	14
1877	....	10.0	10.0	10.0
1878	13.0	10.7	9.7	8.7
1879	8.7	7.1	5.1	3.1
1880	13.6	7.8	5.5	2.6
1881	11.2	6.8	3.5	(—0.3)
1882	8.6	3.2	(—1.0)	(—5.4)
1883	14.0	5.1	1.0	(—0.0)
1884	27.2	20.1	15.2	13.2
1885	15.3	22.8	17.1	14.1
1886	11.5	21.6	15.1	11.2
1887	9.1	18.1	10.8	6.0
1888	8.3	13.9	5.8	0.1
1889	9.4	10.9	2.0	(—4.5)
1890	12.9	13.6	3.2	(—0.6)
1891	14.5	13.1	3.4	0.5
1892	14.4	15.6	4.5	0.4
1893	11.7	14.8	3.1	(—1.9)
1894	8.8	11.2	(—1.2)	(—5.2)
1895	9.4	8.3	(—3.6)	(—4.6)
1896	7.7	3.8	(—5.3)	(—6.3)
1897	10.2	1.9	(—2.8)	(—3.8)
1898	7.7	(—2.5)	(—5.3)	(—6.3)
1899	14.0	2.0	1.0	0.0
1900	10.1	0.0	(—1.9)	(—3.9)
1901	14.2	2.2	1.2	0.2
1902	6.0	(—3.9)	(—5.8)	(—7.8)
1003	10.3	(—1.7)	(—2.7)	(—3.7)
1904	9.0	(—3.0)	(—4.0)	(—5.0)
1905	14.9	2.9	1.9	0.9
1906	16.6	7.4	5.4	3.5
1907	13.9	9.1	6.1	3.3
1908	11.7	8.5	4.6	0.9
1909	24.6	20.8	16.1	11.5
1910	13.3	21.5	15.9	10.5
1911	16.8	25.7	19.2	13.0
1912	17.5	30.0	23.1	16.1
1913	11.5	28.6	20.9	13.1
1914	20.1	30.0	27.4	18.7
1915	12.6	29.7	26.2	16.7
1916	18.7	30.0	30.0	20.9
1917	19.6	30.0	30.0	25.9
1918	13.0	30.0	29.1	24.1
1919	10.3	27.4	25.5	19.7
1920	21.1	30.0	30.0	26.2
1921	19.5	30.0	30.0	30.0
1922	16.9	30.0	30.0	30.0

receives water, by over 2 000 000 acre-ft. when the area estimated to be irrigable in the near future receives water, and by about 5 000 000 acre-ft. when the area estimated to be irrigable in the distant future receives water. Furthermore, it cannot reasonably be assumed that the shortage will be prorated fully over the lands below Boulder Canyon that now receive a full water supply and require 3 000 000 to 4 000 000 acre-ft. for irrigation. The water shortages for new lands below Boulder Canyon will therefore be greatly increased over the percentages indicated in Table [11]. If irrigation development below Boulder Canyon is based on the theory that a flow of 14 000 000 acre-ft., or even of 13 000 000 acre-ft., can be maintained annually, years are indicated for the distant future when not only would there be no water for new lands, but lands now irrigated would receive a scant supply regardless of whether the storage capacity available be 10 000 000, 20 000 000, or 30 000 000 acre-ft. The data indicate that it is unsafe at this time to plan development on the basis of an annual flow of more than between 11 000 000 and 12 000 000 acre-ft.

"The data in Tables 7, 8, 9, and 10 are summarized, as follows:

	Storage Capacity, in Millions of Acre-Feet.			
	5.	10.	20.	30.
Annual Demand of 10 000 000 Acre-Feet:				
Shortage, percentage of time	2	0	....	....
Maximum shortage, percentage	7	0	....	....
Overflow, percentage of time	64	62	....	....
Annual Demand of 11 000 000 Acre-Feet:				
Shortage, percentage of time	16	7	0	....
Maximum shortage, percentage	30	under 20	0	....
Overflow, percentage of time	60	49	31	....
Annual Demand of 11 500 000 Acre-Feet:				
Shortage, percentage of time	....	13	....	....
Maximum shortage, percentage	....	33	....	....
Overflow, percentage of time	....	33	....	....
Annual Demand of 12 000 000 Acre-Feet:				
Shortage, percentage of time	....	18	11	9
Maximum shortage, percentage	....	36	36	32
Overflow, percentage of time	....	28	31	16
Annual Demand of 12 500 000 Acre-Feet:				
Shortage, percentage of time	....	22	....	....
Maximum shortage, percentage	....	38	....	....
Overflow, percentage of time	....	26	....	....
Annual Demand of 13 000 000 Acre-Feet:				
Shortage, percentage of time	....	27	22	22
Maximum shortage, percentage	....	45	41	41
Overflow, percentage of time	....	22	16	11
Annual Demand of 14 000 000 Acre-Feet:				
Shortage, percentage of time	....	....	31	31
Maximum shortage, percentage	....	....	55	55
Overflow, percentage of time	....	....	11	4

"The storage capacity adequate to maintain reasonably uniform annual flow is difficult to determine. It is evidently greater than 5 000 000 and pretty certainly as great as 10 000 000 acre-ft. and less than 20 000 000 acre-ft. Capacity of 10 000 000 acre-ft. will provide annual flow of 11 500 000 acre-ft. with somewhat less onerous conditions of shortage than if capacity of 20 000 000 acre-ft. were utilized to provide annual flow of 12 000 000 acre-ft.

"Increase of Storage Capacity from 10 000 000 Acre-Ft. to the Available Annual Water Supply.

"According to estimates of Bureau of Reclamation engineers the cheapest known storage capacity of 10 000 000 acre-ft. would be the top 65 ft. (approxi-

mately) of the large Boulder Canyon reservoir and would cost about $10 000 000. At $1 per acre-foot of storage capacity the increase in storage capacity from 10 000 000 to 20 000 000 acre-ft. would cost more than $20 per acre-foot of increase in mean annual flow. This would be a doubtful investment if the increase in possible power and irrigation development at and below Boulder Canyon be alone considered, but may be justifiable if the additional storage capacity be provided up stream so that the additional flow could be utilized for power purposes through the much greater head available, in the canyon section of Colorado River.

"It is concluded that storage capacity in excess of 10 000 000 acre-ft. should not be provided, if at all, until the conditions of water supply under future development are much better known than at present.

"Effect of Estimates of Water Supply and Practicable Storage Control on Possibilities of Irrigation Development

"The estimates of water supply and practicable storage for the period 1878-1922 indicate that through long periods not to exceed 12 000 000 acre-ft. of water a year may be relied on for future irrigation development above Laguna Dam and for present and future development below that point.

"Compilation of irrigable areas in the basin of Colorado River and the adjacent basin of Salton Sea, classified by degree of feasibility, has been made by engineers of the Bureau of Reclamation. These areas and estimated net consumption of water by them and by probable diversions from the basin are set forth in the following table [Table 11]:

"TABLE 11.—Irrigable Areas and Water Requirements.

(Areas in Thousands of Acres and Shown to Nearest 10 000. Water Consumption in Thousands of Acre-Feet and Shown to Nearest 100 000.)

	Present:		Immediate Future:		Near Future:		Distant Future:		Total:	
	Area.	Water.	Area.	Water	Area.	Water.	Area.	Water.	Area.	Water.
Above Boulder......	....	....	770	1 300	1 060	2 000	970	1 900	2 900	5 200
Between Boulder and Laguna.......	....	....	170	500	50	200*	870	3 900	1 090	4 600
Below Laguna in United States....	470	2 100	430	1 900	....	....	....	....	900	4 000
Total in United States............	470	2 100	1 370	3 700	1 110	2 100	1 840	5 800	4 990	13 600
In Mexico..........	190	900	300	1 300	310	1 400†	200	900	1 000	4 500
Grand total....	660	3 000	1 700	5 000	1 410	3 500	2 040	6 600	5 810	18 100

"* Includes a little over 800 000 acres listed in available areas by Bureau of Reclamation engineers, but excluded from their estimates of water consumption with the following statement: The Parker-Gila Valley Project is not considered fully as feasible as other Class C projects in either basin and as available water supply is insufficient for the inclusion of its entire area at the assumed diversion duty, the area of this project is reduced to fit the water supply.

"† This area not dependent on flow of Colorado River at Laguna as it may be served by return flow, waste, etc., near head of Gulf.

"The foregoing table [Table 11] does not include 1 500 000 acres of land now irrigated above Laguna Dam, requirements of which have been provided for and excluded from the estimates of water supply.

"The conclusion is obvious that the water supply is inadequate to provide for a very large area in the projects that have been classified as becoming feasible only in the distant future. However, before such projects are seriously

considered for development, far more and better information as to water supply will be available and a reasonably accurate estimate of the adequacy or inadequacy of the water supply available for them can be made. An attempt to solve all the problems of Colorado River at a single stroke on the basis of the meager information now available is likely to result in ill-advised expenditure if nothing worse."

Storage Required at Present for Irrigation of Lower Basin

The entire low-water flow of the Colorado in the Lower Basin is now appropriated, and storage will be necessary for any expansion of acreage and a small amount of storage should be provided to take care of present acreage in years of less than average water supply. The "Fall-Davis" report (page 40) shows that, in the year of smallest run-off, 2 340 000 acre-ft. of storage would be sufficient to provide for the feasible acreage in both the Upper and the Lower Basins. Since the "Fall-Davis" report was issued, the Bureau of Reclamation has made extensive studies of possible irrigation in the Colorado Basin, the net results of which indicate that all lands likely to be irrigated by 1940 in both the Upper and the Lower Basins can be provided for by 2 300 000 acre-ft. of storage in the worst year of record. Aside from 1902, which was the worst year, 1 500 000 acre-ft. of storage would have been sufficient.

About 840 000 acre-ft. of storage will take care of all development contemplated in the "Swing-Johnson" Bill, including that in Mexico supplied by the Imperial Canal, for years like 1902, and 500 000 acre-ft. would be sufficient for all other years of record. If 4 000 000 acre-ft. of storage is provided for flood control, it can be operated in such manner as to take care of irrigation needs in the Lower Basin as far as it is profitable to do so at this time.

Power

The power demand in Southern California has increased in the past 14 years at a rate of about 13% compounded annually. There is a present mining load in Arizona carried by steam power that justifies the belief that 75 000 to 80 000 kw. could be absorbed from the Colorado as soon as it is available, and the development of power on the Colorado at Diamond Creek would now be under way but for lack of Federal authority to use the public lands necessary. There are mining possibilities in Nevada and Southern Utah that may create a large demand. They are too speculative at present to justify expenditure for power development, but may greatly accelerate such development once it is started. The mining load in the vicinity of Salt Lake City has grown so that development of the Flaming Gorge site on the Green River, just below the Wyoming line, is now under serious consideration by the Utah Power and Light Company.

The use of power in the United States has grown with surprising rapidity. When it is considered that the use of electrical power commenced about 30 years ago, it is not extravagant to conclude that all the power in the Canyon Section of the Colorado may be absorbed in the next 25 or 30 years. Although immediate expenditure should be limited to immediate needs, the possibility of such a demand for power is certainly great enough to justify taking it into account in planning the development of the Colorado, to the extent of insur-

ing the ultimate development of the greatest possible power from the Canyon Section commensurate with the higher uses of the water. This means that projects should be planned to use all available head, that storage for equalization of flow should be provided above the Canyon Section, and that storage for re-regulation of flow for irrigation in the Lower Basin should be provided at the bottom of the Canyon Section. Storage located at the bottom of the Canyon Section for flood protection of the Lower Basin does not conform to the requirement that storage for equalization of flow should be above the Canyon Section. It should be built, therefore, only to the minimum capacity that will give relief, especially as storage above the canyon may be relied on to give additional relief in the near future, and it should be located, if practicable, so that later it will provide for re-regulation of flow for irrigation in the Lower Basin.

Aside from the Flaming Gorge and Diamond Creek developments on the Colorado, any developments of power in the immediate future must depend on Southern California for a market, and the size of the development should be limited to what that market can absorb with certainty, in order to keep the carrying charges at a minimum during the periods of construction and loading.

Table 12 has been compiled from information in the files of the Federal Power Commission to show the possible hydro-electric development in Southern California.

The projected developments of the San Joaquin Light and Power Company are on the North Fork of Kings River. Expenditures on these works have been commenced on a scale that will probably make necessary their completion in spite of the fact that their estimated cost is more than $200 per horsepower installed.

The projected developments of the Southern California Edison Company will probably go forward up to 1935. The installed capacity for that year includes what are known as the West Side Developments of the San Joaquin River. They will be expensive, costing probably more than $200 per horsepower installed, and may be deferred if Colorado River power becomes available.

The proposed developments of the City of Los Angeles are in two regions, namely, Owens Valley and South Fork of Kings River. The developments in the latter region, at least, may be deferred if Colorado power becomes available.

Table 13 shows the power generated in Southern California by the four principal producers. Their combined load has grown at an average rate of about 13% compounded annually. Nearly 80% of the power has been hydropower and has operated at an annual capacity factor of from 45 to 55 per cent.

Table 14 shows predictions of growth of load in Southern California. The prediction* made by Frederick H. Fowler, Assoc. M. Am. Soc. C. E., is stated by him to be the probable maximum. The prediction† made by A. H. Markwart, M. Am. Soc. C. E., was for the entire State of California and is probably

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 836.

† *General Electric Review*, 1922.

TABLE 12.—CALIFORNIA HYDRO-POWER PROJECTS.

Companies.	Streams.	September 1921, installed capacities, in kilovolt-amperes.	PROGRAM OF DEVELOPMENT IN KILOVOLT-AMPERES:												Ultimate capacity of new projects.
			1922	1923	1924	1925	1926	1927	1928	1929	1930	1935	1940	1945	
San Joaquin	Kings River No. 175	82 450					15 000	15 000		15 000	15 000	30 000	60 000	147 000	297 000
So. Calif. Edison	Big Creek No. 67	201 350	30 000	16 000		25 000									71 000
"	San Joaquin			75 000		16 000		99 000	99 000		195 000	212 000			696 000
Nev.-Calif. Power	Leevining, Rush,	40 100		2 800											2 800
	Owens, and Snow					3 000	7 500				12 500				23 000
So. Sierras	Creeks					2 000	1 500								3 500
City of Los Angeles		72 460							17 500	37 500	33 450	46 080	126 220	394 000	654 750
San Gorgonio	San Gorgonio				2 700										2 700
Sespe Light and Power	Piru Creek						6 000				7 000				13 000
Merced Irrig.	Merced River No. 88					30 000									30 000
Fletcher, Edison	Boulder Creek										980				980
Total		396 360	30 000	93 800	2 700	76 000	30 000	114 000	116 500	52 500	263 930	288 080	186 220	541 000	1 794 730
Accumulative total				123 800	126 500	202 500	232 500	346 500	463 000	515 500	779 430	1 067 510	1 253 730	1 794 730	
			426 360	520 160	522 860	598 860	628 860	742 860	859 360	911 860	1 175 790	1 463 870	1 650 090	2 191 090	

low for Southern California. Column (3) of Table 14 which was compiled from Mr. Fowler's prediction on the assumption that hydro-developments will continue to furnish 80% of the power and will operate at an annual capacity factor of 55%, would be high as a basis for immediate investment if it were not for the fact, as mentioned previously, that some of the California installation may be deferred in favor of Colorado development. Column (5) of Table 14 contains the totals from Table 12. A comparison with Column (3) shows that California developments will probably take care of the growth of load in Southern California up to 1935. At that time the growth of load will call for increased installation of about 100 000 kw. per year.

TABLE 13.—POWER GENERATED IN SOUTHERN CALIFORNIA.*

(Systems Included: Southern California Edison Company; Southern Sierras Power Company; San Joaquin Light and Power Corporation; and City of Los Angeles.)

Year.	HYDRO:		STEAM:		TOTAL:		Percentage hydro.
	Kilowatt-hours.	Percentage increase	Kilowatt-hours.	Percentage increase.	Kilowatt-hours.	Percentage increase.	
1910	324 331 916	....	124 181 024	....	448 512 940	....	72.3
1911	376 988 076	16.2	162 675 931	30.6	539 664 007	20.33	69.8
1912	383 136 890	1.6	273 354 263	68.0	656 491 153	21.64	58.35
1913	431 510 017	12.6	307 293 993	12.45	738 804 010	12.53	58.40
1914	740 991 812	71.7	104 288 601	—66.26	845 780 413	14.47	87.61
1915	788 344 373	6.4	93 557 059	—10.29	881 901 432	4.10	89.39
1916	876 636 452	11.11	63 578 530	—32.05	940 214 982	65.70	93.23
1917	924 781 935	5.4	165 909 200	160.90	1 090 691 135	16.00	84.78
1918	1 005 383 515	8.7	250 108 913	50.75	1 255 492 428	15.10	80.07
1919	986 852 219	—1.8	414 084 672	65.56	1 400 936 891	11.58	70.44
1920	1 161 522 861	1.76	432 753 084	4.50	1 594 275 945	13.80	72.85
1921	1 514 873 779	30.5	314 532 165	—27.32	1 829 405 944	14.74	82.08
1922	1 848 494 288	22.0	123 615 130	—60.7	1 972 109 418	7.80	93.73
1923	1 992 877 000	7.8	406 469 000	22.88	2 399 346 000	21.66	83.05
Total..	12 356 725 133	15.0†	3 236 401 698	9.5†	15 593 126 698	13.8†	79.8

* Compiled from *Water Supply Paper No. 493*, U. S. Geological Survey, and records of U. S. Geological Survey.

† Compound rate of increase for entire period.

A Colorado River power development probably cannot be completed before 1930, but should be completed by 1932. In view of the above, the estimates of power company engineers that a Colorado River development planned now should not exceed an installed capacity of 300 000 kw. appears to be sound. A larger development will ultimately be absorbed, but the risk of excessive carrying charges is not justifiable if the smaller development is feasible. It has become the custom to talk in such large figures concerning the Colorado that it may be pertinent to point out that the only hydro-electric developments in this country as large as 300 000 kw. are at Niagara Falls and Muscle Shoals, and that the total installed capacity in Southern California at present is about 500 000 kw.

POSSIBLE STORAGE RESERVOIRS

Table 15, prepared by the U. S. Bureau of Reclamation, gives information on reservoir sites on the upper tributaries.

TABLE 14.—PROBABLE POWER DEMAND OF SOUTHERN CALIFORNIA.

Year.	Per Fowler: Increase at: 13% to 1925 12% to 1930 10% to 1935 8% to 1940 in millions of kilowatt-hours. (1)	Per Markwart: Increase at: 11.1% to 1930 7.2% to 1940 4.1% to 1950 in millions of kilowatt-hours. (2)	Fowler's Estimate: Assume hydro produces 80% of kilowatt-hours and operates at 55% capacity factor. Required installed capacity, in kilowatts. (3)	Markwart's Estimate: Hydro produces 90% of kilowatt-hours and operates at 72% capacity factor. Required installed capacity, in kilowatts. (4)	Possible installed capacity of Southern California hydro per Table 12. (5)
1924	2 710	2 662	451 660	384 400	522 860
1925	3 062	2 954	510 330	435 700	598 860
1926	3 429	3 280	571 490	483 900	628 860
1927	3 840	3 640	640 000	537 000	742 860
1928	4 301	4 040	716 820	596 000	859 360
1929	4 817	4 484	802 820	661 600	911 860
1930	5 395	4 975	899 150	733 900	1 175 790
1931	5 985	5 109	991 510	758 700	....
1932	6 528	5 477	1 088 000	808 000	....
1933	7 181	5 879	1 196 810	867 700	....
1934	7 899	6 302	1 316 470	930 000	....
1935	8 689	6 755	1 448 140	996 700	1 463 870
1936	9 384	7 241	1 563 970	1 068 000	....
1937	10 134	7 762	1 689 000	1 145 000	....
1938	10 944	8 369	1 823 970	1 234 000	....
1939	11 819	8 971	1 969 800	1 323 000	....
1940	12 764	9 617	2 127 300	1 419 000	1 650 090
Extended at 7% 1941	13 657	10 011	2 276 130	1 477 000	....
1942	14 613	10 421	2 435 460	1 537 000	....
1943	15 635	10 848	2 605 800	1 600 000	....
1944	16 729	11 292	2 788 120	1 666 000	....
1945	17 890	11 754	2 981 610	1 734 000	2 191 090

Of these sites, it may be stated that:

1.—The Ouray site will be expensive to develop, because of depth to bed-rock and width between abutments.

2.—The Kremmling site is probably unavailable on account of interference with the Denver and Salt Lake Railroad.

3.—The Bedrock site commands a relatively small run-off.

TABLE 15.

Stream.	Reservoir site.	Raise in water surface, in feet.	Reservoir capacity, in acre-feet.	Average annual run-off at site, in acre-feet.	FOUNDATION CONDITIONS: Tests by	Depth to bed-rock, in feet.
Green	Flaming Gorge..	240	4 000 000	2 300 000	Drilling	73
"	Ouray	210	16 000 000	5 200 000	"	121
Yampa.......	Juniper..........	200	1 550 000	1 200 000	"	24
Colorado....	Kremmling.......	230	2 200 000	1 200 000	"	104
"	Dewey...........	215	2 270 000	6 800 000	"	44
Dolores......	Bedrock.........	210	800 000	400 000	None	....
San Juan...	Bluff............	206	1 350 000	2 300 000	"	....

The U. S. Bureau of Reclamation estimates the cost of developing four of these sites as shown in Table 16.

On the basis of cost as shown on Table 16 the average cost will be about $4.75 per acre-foot.

TABLE 16.

Reservoir.	Capacity, in acre-feet.	Cost.
Flaming Gorge....................	3 120 000	$16 000 000
Juniper...........................	1 550 000	4 000 000
Dewey.............................	2 270 000	11 200 000
Bluff.............................	1 400 000	8 800 000
Totals..........................	8 340 000	$40 000 000

The Flaming Gorge site is now under permit to the Utah Power and Light Company and will probably be developed in the near future.

The Juniper and Dewey sites can be developed for about 4 000 000 acre-ft. at a cost of about $4 per acre-foot.

In addition to these sites, there are possible reservoir sites at Glenn Canyon, Boulder Canyon, and Mohave, the capacities and areas of which are shown on Fig. 12.

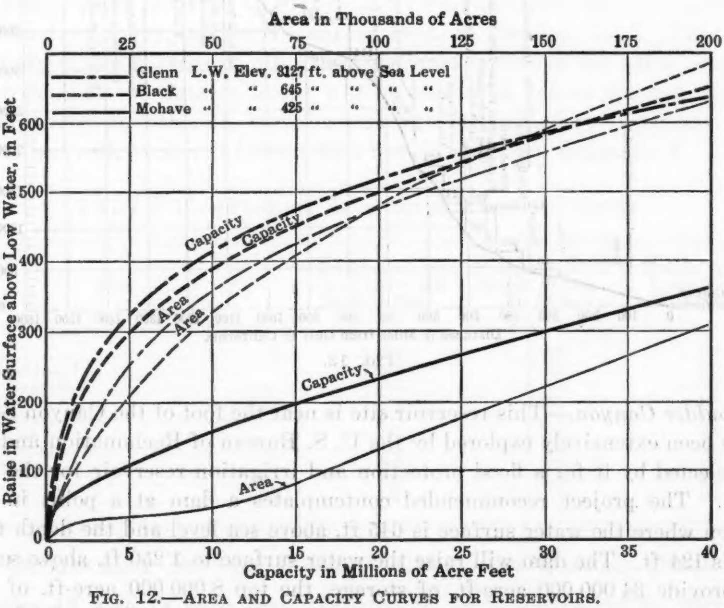


FIG. 12.—AREA AND CAPACITY CURVES FOR RESERVOIRS.

Glenn Canyon.—This reservoir is near the head of the Canyon Section of the main river. The Southern California Edison Company explored the Glenn Canyon site and found bed-rock at a maximum depth of 80 ft. The walls and bed of the canyon at the dam site are massive red sandstone, and the canyon walls rise almost vertical to heights of 1 000 to 1 500 ft. The engineers of the U. S. Bureau of Reclamation state that the sandstone is so poorly cemented that its safe bearing power will not be much more than 20 tons per sq. ft. Three samples of core in compression tests failed at 2 220, 12 900 and 10 480 lb. per

sq. in., respectively. The strength in compression of the weakest sample tested is, therefore, greater than that of mass concrete and, contrary to the views of the engineers of the U. S. Bureau of Reclamation, it seems to be safe to assume that the maximum stresses will be determined by the concrete and not by the bearing power of the stone in the foundations and abutments. The greatest objection to the site is its great distance (about 135 miles) from a railroad.

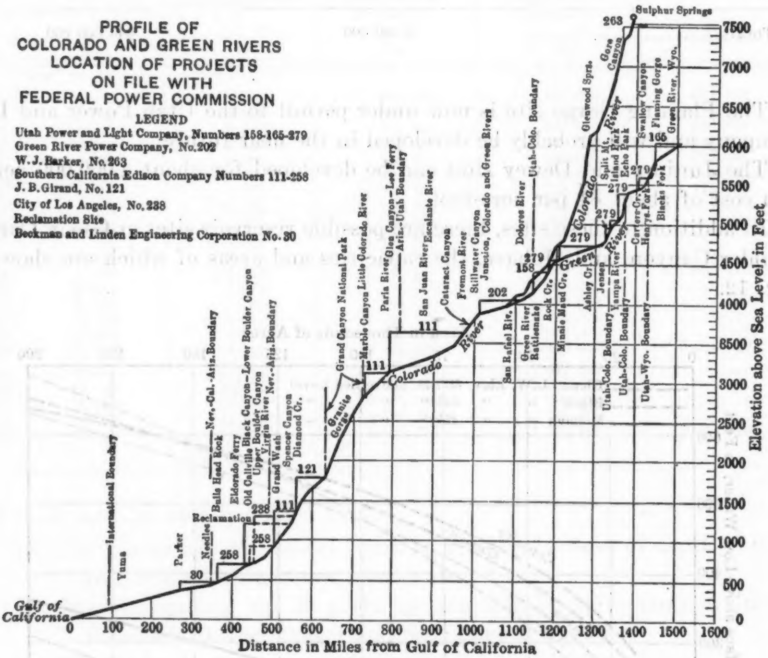


FIG. 13.

Boulder Canyon.—This reservoir site is near the foot of the Canyon Section. It has been extensively explored by the U. S. Bureau of Reclamation and is the site selected by it for a flood protection and irrigation reservoir for the Lower Basin. The project recommended contemplates a dam at a point in Black Canyon where the water surface is 645 ft. above sea level and the depth to bed-rock is 124 ft. The dam will raise the water surface to 1 250 ft. above sea level and provide 34 000 000 acre-ft. of storage, the top 8 000 000 acre-ft. of which are to be reserved for flood protection and irrigation of the Lower Basin. The dam will develop 660 000 h. p. continuously with an average head of 550 ft. This project will be further discussed under the head of "Development Between Needles, Calif., and Diamond Creek".

Mohave Valley.—The Mohave Valley site is about 100 miles nearer the Lower Basin than Boulder Canyon and presents many other advantages as a site for flood protection and re-regulation for irrigation in the Lower Basin. It is easily accessible, develops great capacity with a relatively low dam, and leaves the entire Canyon Section of the main river free to be developed for

power. If satisfactory foundations are found, the only objection to the site is that its flowage damages may be high, as it will require moving the Town of Needles, a division point on the Santa Fé Railroad, and re-locating 15 to 20 miles of main-line railroad.

The U. S. Bureau of Reclamation has estimated the cost of providing 10 000 000 acre-ft. of storage at the previously mentioned sites, as follows:

Glenn Canyon	$77 500 000
Boulder Canyon	27 000 000
Mohave Valley	26 000 000*

It also estimates that for a reservoir of 34 000 000 acre-ft. at Boulder Canyon, the 8 000 000 acre-ft. reserved for flood protection would cost $12 000 000, if the total cost were pro-rated on the basis of capacity used.

The data on which the estimates for Glenn Canyon, Mohave Valley, and the upper reservoirs are based, are so incomplete that they constitute a doubtful basis for determining relative costs.

Possible Dam Sites on the Colorado

In 1923, a party from the U. S. Geological Survey went down the Colorado by boat from Lees Ferry, or Glenn Canyon, Ariz., to Needles, Calif. This party made topographic surveys of sites that appeared favorable for dams, and it is understood that the data collected will be published before the end of 1924. The list, shown in Table 17, includes some sites examined by this party. The depth to bed-rock at most of these sites has not yet been determined.

TABLE 17.—Possible Dam Sites, Colorado River.

Site.	Elevation of low water above sea level, in feet.	Probable limit of height of dam, in feet.	Distance, in miles, from Gulf of California.
Diamond	1 335	1 800	567
Bridge Canyon	1 207	2 000	557
Spencer Canyon	1 105	1 700	547
Devils Slide	1 034	1 330	537
Pierce Ferry	905	1 100	516
Boulder Canyon	705	1 250	459
Black Canyon	645	1 250	438
Bulls Head	496	650	372
Mohave	425	605	318
Laguna	151	......	135

Development Between Needles, Calif., and Diamond Creek

Tables 18 and 19 have been prepared to show the extent to which the Boulder Canyon Project of the U. S. Bureau of Reclamation fails to conform to a full development of the Colorado River below Diamond Creek.

In Table 19, Plan 1 is the Boulder Canyon Project. It is contemplated that when further irrigation is developed in the Lower Basin, a re-regulating dam will have to be constructed at Bulls Head.

Plan 2 includes the Bulls Head Dam. The head between the Boulder Project and Diamond Creek (about 80 ft.) is too small to permit of economic

* Includes $13 000 000 for flowage damage.

TABLE 18.—DATA ON BOULDER CANYON AND MOHAVE RESERVOIRS.

BOULDER CANYON RESERVOIR.

Maximum elevation of pool above sea level, in feet.	Minimum elevation of pool above sea level, in feet.	Height of dam above low water, in feet.	Mean head available for power, in feet.	Storage available for flood protection, in acre-feet.	Mean area of reservoir exposed to evaporation, in acres.	Evaporation loss per year at 5 ft, in acre-feet.	Present evaporation loss, in acre-feet.	Increase of evaporation loss, in acre-feet.	Acres that could be irrigated with water lost.	Horse-power available, (mean head x 1 210).
905	905	260	260	0	35 000	175 000	35 000	140 000	31 000	305 000
905	905	345	345	0	37 000	185 000	40 000	145 000	32 000	418 000
1 030	1 030	385	385	0	68 000	340 000	40 000	300 000	67 000	466 000
1 250	1 250	605	605	0	160 000	800 000	50 000	750 000	167 000	732 000
1 250	1 155	605	550	8 000 000	130 000	650 000	50 000	600 000	133 000	667 000
1 250	1 225	605	580	4 000 000	145 000	725 000	50 000	675 000	150 000	702 000
1 250	1 235	605	590	2 350 000	155 000	775 000	50 000	725 000	161 000	714 000

MOHAVE RESERVOIR.

Maximum elevation of pool above sea level, in feet.	Minimum elevation of pool above sea level, in feet.	Height of dam above low water, in feet.	Mean head available for power, in feet.	Storage available for flood protection, in acre-feet.	Mean area of reservoir exposed to evaporation, in acres.	Evaporation loss per year at 5 ft, in acre-feet.	Present evaporation loss, in acre-feet.	Increase of evaporation loss, in acre-feet.	Acres that could be irrigated with water lost.	Horse-power available, (mean head x 1 210).
425	......	....	....	....	....	....	....	....	....	....
525	425	100	....	4 340 000	68 000	340 000	310 000	30 000	7 000	0
525	470	100	....	4 000 000	71 000	355 000	310 000	45 000	10 000	0
545	425	120	....	6 125 000	68 750	343 750	310 000	33 750	7 500	0
545	490	120	80	4 000 000	72 800	364 000	310 000	54 000	12 000	96 000
582	425	157	....	10 000 000	88 000	440 000	350 000	90 000	20 000	0
605	425	180	....	12 800 000	97 000	485 000	350 000	135 000	30 000	0
605	530	180	125	8 000 000	98 000	490 000	350 000	140 000	31 000	131 000
605	545	180	155	4 000 000	113 000	565 000	350 000	215 000	48 000	187 000

NOTE.—Average evaporation loss, Topock to Yuma, for 1922–23, 1 770 000 acre-ft.; overflowed area, 306 000 acres. Overflow area: Mohave Valley, 62 500 acres; Cottonwood Valley, 9 500 acres; total, 72 000 acres. Present evaporation loss may be taken as: Mohave Valley, 310 000 acre-ft.; Cottonwood Valley, 40 000 acre-ft.; total, 350 000 acre-ft.

TABLE 19.—COMPARATIVE PLANS OF DEVELOPMENT OF COLORADO BELOW DIAMOND CREEK

(Assuming Up-River Storage Developed. Evaporation Loss at 5 Ft. per Year; Irrigation Duty, 4 to 5 Ft. per Year.)

Plan.	Dam.	Elevation of pool above sea level, in feet.	Head available for power, in feet.	Storage available for flood and irrigation in acre-feet.	Evaporation loss, in acre-feet.	Horse-power available.	Acres lost on account of evaporation.	Cost.
			Bureau of Reclamation Plan.					
1	Boulder	1 250	550	8 000 000	600 000	637 000	133 000	$50 000 000
2	Bulls Head	645	0	1 650 000	25 000	0	....	14 000 000
	Boulder	1 250	590	2 350 000	725 000	714 000	....	50 000 000
	Totals	......	590	4 000 000	750 000	714 000	169 000	64 000 000
3	Mohave	605	125	8 000 000	140 000	132 000	....	27 000 000
	Boulder	1 030	385	....	300 000	465 000	....	27 000 000
	Devils Slide	1 330	300	....	10 000	363 000	....	22 000 000
	Totals	......	810	8 000 000	450 000	960 000	100 000	76 000 000
4	Mohave	605	180	4 000 000	215 000	187 000	....	27 000 000
	Boulder	1 030	385	....	300 000	465 000	....	27 000 000
	Devils Slide	1 330	300	....	10 000	363 000	....	22 000 000
	Totals	......	865	4 000 000	525 000	1 015 000	117 000	76 000 000
5	Mohave	545	0	6 000 000	33 750	0	....	24 000 000
	Black Canyon (Lower End)	905	345	....	145 000	418 000	....	26 000 000
	Pierce Ferry	1 112	207	....	5 000	248 000	....	17 000 000
	Spencer Canyon	1 330	218	....	5 000	262 000	....	18 000 000
	Totals	......	770	6 000 000	183 750	928 000	38 000	$85 000 000

development, so that Plan 2 represents full development under the Reclamation plan.

Plans 3 and 4 contain similar structures, namely, a dam 180 ft. high at Mohave, a dam 387 ft. high at the Reclamation site in Boulder, and a dam 298 ft. high at Devils Slide. Plan 3 is comparable to Plan 1 and Plan 4 to Plan 2. The comparison shows that the Reclamation plan will cost $12 000 000 less, but will ultimately prevent the development of about 300 000 h. p. and 50 000 acres of land. The magnitude of this loss of power may be made clearer by pointing that the hydro-power generated in Southern California in 1923 was just about 300 000 h. p. If applied for under the Federal Water Power Act, the Reclamation project would be refused a license until it was modified to conform to a full scheme of development. It should not be built by the Federal Government on the lines proposed.

Plan 5 is presented in order to illustrate the limit in curtailment of evaporation losses.

It should be stated in connection with the figures used in the comparative plans just mentioned, that power and evaporation losses of the Boulder Project are believed to be low and complete data probably will modify estimates of cost so as to wipe out more or less the difference of $12 000 000 and, as a flood protection and irrigation project, Mohave has so many advantages that it should be fully investigated before any other project is adopted. If it is found feasible, a site should be sought at the lower end of Black Canyon for a power dam sufficiently high to back water up to either Pierce Ferry or Devils Slide, and the relative feasibility of the two latter sites should determine the height to which the dam should be built.

Silt

The Colorado carries large quantities of silt. If a flood-control and irrigation dam at the lower end of the Canyon Section were the only one in prospect, large capacity would have to be provided for silt. With the Diamond Creek Dam ready to be built for power and other power dams sure to follow before Diamond Creek can become filled, there is no justification for a large increase in present expenditure to provide silt capacity. If the Mohave Reservoir is built to a capacity of 6 000 000 acre-ft., it will give liberal capacity for silt deposit.

Increase of Low-Water Flow Undesirable at Present

There is serious objection on the part of the upper States to a material increase of the low-water flow of the river until the Colorado Compact is ratified by Arizona, the only State which has not yet done so. This compact was negotiated for the purpose of determining by mutual agreement, rather than by litigation, the allocation of waters between the several States in the Basin. There is good basis for objection to the Boulder Project of the U. S. Bureau of Reclamation, because it would put to beneficial use for power practically all the water in the river and thereby acquire prior rights that might have to be purchased before any further irrigation up stream could be accomplished.

There would be little ground for objection if the project were reduced to provide only for present needs of flood protection and irrigation.

The Boulder Project would provide water in the Lower Basin at all seasons far in excess of present irrigation requirements. This water will pass into Mexico and there be used for irrigation. Once used, its withdrawal for use in the United States will be difficult, if not impossible. As heretofore shown, storage for flood protection will not obviate the need for maintaining levees and bank protection in Mexico, and a treaty to facilitate this work and to limit the quantity of water to be supplied Mexico should be negotiated before the quantity for irrigation in Mexico is increased.

Work Which Should Be Undertaken by the United States

There are now before the Federal Power Commission applications covering practically the entire Canyon Section of the river. (See Fig. 13.) All the development justified at present, including flood control and irrigation, will be undertaken by private capital under adequate Federal and State regulation if the Federal Water Power Act is left free to function.

If, notwithstanding this situation, Congress deems the special conditions on this stream such as to demand Federal construction of a flood protection reservoir, investigation of the Mohave site should be made at once and, if reasonable foundations can be found, the dam should be built there. In view of the objections to increasing the low-water flow at present, it would be best to construct this dam for purposes of flood control and regulation of flow for irrigation only and not for the development of power. It could then be operated so as to prevent the appropriation and use of water beyond present needs. If power is installed, there will be great pressure to operate for the power demand. There seems to be small justification for charging the entire cost of such a dam to the power consumers but, if Congress decides to do so, a dam 180 ft. high at Mohave will produce more than 130 000 h. p., which would justify an installation of about 240 000 h. p. Projects in Southern California have averaged about $130 per horse-power installed, but there are few developments left that can be built for less than $200 per horse-power installed. The estimated cost of the Mohave Dam is $27 000 000, which charged to 240 000 h. p., gives less than $115 per horse-power. To this, add $35 per horse-power for cost of power equipment, and the total is $150 per horse-power installed. This price compares so favorably with prospective California developments that there is little doubt that the power can be made to carry the cost of the dam without raising present prices to consumers. If the Mohave Dam proves to be feasible, it can be built in 4 years as against the 8 years estimated for Boulder Canyon Dam.

Conclusion

The proposed high dam in Boulder Canyon should not be built because it will curtail ultimate power development by 300 000 h. p. and ultimate irrigation by about 50 000 acres.

All the development needed on the Colorado will be built by private capital under adequate Federal and State regulation if the river is given over to development under the Federal Water Power Act.

If the Federal Government decides to provide flood storage for the Lower Basin, it should be provided in Mohave Valley where it will conform to the maximum use of the river for all purposes. Such a dam should be designed and operated for the present in a manner that will not increase the low-water flow beyond present irrigation needs. Steps should be instituted as early as practicable to negotiate a suitable treaty with Mexico covering the maintenance of levees, canals, etc., in Mexico and an agreement as to the water that will be provided for Mexican lands.

WATERWAY AND RAILWAY EQUIVALENTS

By William M. Black,* M. Am. Soc. C. E.

Synopsis

New waterway lines of transportation should not be established unless an analysis of the existing railway, highway, and waterway lines and of the need for transportation shows that the proposed new line is required and that if established it will produce an annual saving in the cost of transportation greater than the interest on construction plus maintenance and operating costs.

As the cost of all lines of communication is paid for from the public wealth, an exercise of unprejudiced judgment is demanded, before an investment is made. In selecting any one of the general transportation agencies for use, the total tonnage requiring movement must be analyzed as to volume, distance to be moved, continuity of traffic, kinds of commodities, importance of time of transit, class of carriers required, and terminal facilities.

The cost of transportation by any method depends primarily on the cost of moving the required tonnage. In this cost must enter the charges for the interest on the capital cost of the carriers and of the road bed or waterway, for the amortization of the investment, and the cost of maintenance and operation. In this last cost, the proportion due to the time element in operation must be included.

The cost of movement along a waterway is a function of the resistance offered by the water to the movement of a boat; such resistance varies with the speed, with the surface and form of the boat, and with the dimensions of the waterway. The resistance increases much more rapidly than the speed, especially in channels of restricted dimensions.

In forming a project for a waterway, the size and speed for the carriers necessary to meet traffic conditions should first be determined, and from these the dimensions necessary for an adequate waterway. Many experiments have been made in Europe and America in order to determine the resistance to movement of boats of various classes in channels of different dimensions. The formulas deduced are empirical and must be applied with judgment. A number of references are given to authorities on this subject, to de Mas, Grübe, and Haack in Europe, and to Sweet, Taylor, Sadler, and McEntee in America. Probably the most comprehensive series of experiments were those made under the direction of the Engineer Department, U. S. Army, by the United States Tow Boat Board. The Government publications on this subject

Note.—Written discussion on this paper will be closed with the December, 1924, *Proceedings*. When finally closed, the paper, with discussion, will be published in *Transactions*.

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are noted; also the conclusions reached. When a speed exceeding 3 miles per hour is necessary, the form of the boat becomes important, as shown by examples. From the U. S. Navy tank tests, it appears that if, for traffic reasons, a cargo carrier with a load capacity of 2 000 tons, and a speed of 5 miles per hour, is necessary, the minimum channel dimensions should be a depth of 10 ft. and a bottom width of 188 ft.

The writer then discusses the increase in depth required to allow for the squat of boats, and the increases in channel width required at bends and to allow for the oblique position taken by boats and tows subjected to a heavy side wind. Provision must be made to prevent the erosion of banks under the action of waves due to the boat and in restricted waterways due to current velocities actuated under and past the hull.

In the Mississippi Valley waterways, tows are generally pushed by stern-wheel or tunnel towboats; in the Hudson River System, they are pulled by propellers. Tests on the Mississippi River showed that for six barges a formation of two abreast and three deep offered the least resistance.

In studying a project for a waterway all elements entering into costs of transportation must be considered and savings in transportation costs must be weighed against these elements.

Statistics are cited to show that under comparable conditions the power required for propulsion is 3.13 lb. per ton of train weight for railways as contrasted with 1.87 lb. per ton of displacement for waterways; as to carrying capacity during 24 hours, the relation is 144 000 tons of freight for a single-track railway to 884 000 tons for the waterway.

Undoubtedly, transportation is cheaper by adequate waterways than by rail for long runs when the time element is not of primary importance, and quicker as well as cheaper for comparatively short runs. Too little attention, however, has been given in the past to the construction of waterways with reference to the class of carriers required and to their equipment with the most suitable carriers. The relative advantages and relative costs of each medium of transportation should be weighed with care before new projects are undertaken. Destructive competition should cease; co-operation should be required.

Before this country was developed, any improvement of its transportation facilities was advantageous: The opening of a dirt road to take the place of a trail was desirable; the clearing of a watercourse, or the construction of a canal of any navigable kind, formed a beneficial public improvement. To-day, however, almost all sections are provided with transportation facilities such as were beyond the imagination of the colonists so that when new lines of transportation are created, the money and labor expended are wasted unless they are needed by commerce to provide a means of transportation better and at lower cost than the facilities which already exist. Railways with their continually improving equipment, and automobiles, traveling on the smooth, hard roads, carry freight more cheaply than land carriers elsewhere in the world. On the other hand, the general use of our internal waterways as freight carriers has diminished. To-day, if a waterway will not accommo-

date carriers which in safety and cost of operation are superior to the land carriers, the waterway is not used. The old primary importance of internal waterways as a means of transportation has vanished. The expediency of their use for this purpose is now relative, and such use is based on the economy it will produce.

It is a National policy to create and maintain highways and waterway channels at the public expense. This expense, like the cost of railways, must be defrayed from the general wealth of the people. Before new projects for highway or waterway construction are entered into the same care should be exercised as is used before private funds are invested. The cost of the undertaking must be estimated, the facilities it will create must be determined, and the economies which these facilities will produce must be weighed against the legitimate interest charges on the investment, and the cost of maintenance and operation of the work.

In the case of a proposed creation of a new line of communication by water, the first thing to be considered is the need—the traffic that will use it and the decrease in the cost of transportation such use would produce. In considering the traffic, its component commodities must be detailed, also their origin and destination, the importance of the time of transit, the class of carrier most suitable, the number of such carriers required, and the best and most economical means for their propulsion. Consideration must be given to the continuity of the traffic and as to whether it originates from few or from many sources, whether it is destined for few or for many consignees, and whether it is to be carried in bulk or in packages. In considering the probable diminution of the cost of transportation, there must be borne in mind the interest on the capital cost of the new channel and the cost of its maintenance, the capital costs of towboats and barges and of their maintenance, the cost of operation, including that of loading and unloading, terminal charges, and the charges for time laid up, etc., and, in addition, the amortization of the investment.

It is evident that even when there is a real commercial demand for a waterway, such a line of communication will not be a financial success unless the channel available is suitable for navigation by the class of carriers required; and unless the channel facilities permit an inexpensive type of carrier.

Water Resistances.—The cost of movement depends on the resistance offered by the water, this being a function of the surface, form and speed of the carrier, and depth and width of the channel. The resistance of the water, and, consequently, the power required to move the carrier, increases rapidly with the speed, somewhat more than in the proportion of the square of the speed. This is important, since with high wages and high cost of subsistence the time consumed in a voyage forms an important element in operating costs. The increase of power required to produce an increase of speed in shoal water is exemplified by an experiment made in 1911 with the steamer, *Hendrik Hudson,* which is 400 ft. long, with a beam of 45 ft. and a draft of $9\frac{1}{2}$ to 10 ft. Moving in a channel 14 ft. deep and about 900 ft. wide, she attained a speed of 9.1 miles per hour with the exertion of 350 h. p., while to attain a speed of 9.48 miles per hour, 730 h. p. was required.

Toward the close of the Nineteenth Century extended experiments were made in this country and in Europe to determine the resistance developed by boats moving in restricted channels. In the United States, the late Elnathan Sweet, M. Am. Soc. C. E., made experiments on the Erie Canal and proposed a formula for resistance which has been published.* In 1901, Mr. Sweet again made an analysis of the subject† and, in 1902, wrote another paper‡ in which he repeated a part of his report on the Barge Canal. He invites particular attention to the experiments made in Germany by R. Haack, who has evolved an interesting method of determining resistance to traction in canals from the differences in canal level in front of and behind a boat. Mr. Sweet mentions the important effect of these differences of level in producing velocities in the water along the sides and bottom of the canal with consequent erosive action.

Channel Dimensions.—Haack's formula was used in fixing the dimensions of the Panama Canal. The method of application in that instance, together with two plates showing, respectively, the curve of depression of water surface for varying ratios of ship and canal cross-sections and the speed curves for restricted channels, have been published.§ Later, Professor Sadler, of the University of Michigan, and Admiral Taylor and other officers of the U. S. Navy, made many tests of model boats in tanks and deduced useful formulas. Other tests with models and with full-sized carriers were made under the direction of the U. S. Engineer Department by the U. S. Tow Boat Board.‖ A report of model experiments with canal barges in canals of varying sizes was made in 1917 and 1918, by Constructor William McEntee, U. S. N., and a discussion of the subject by Colonel F. R. Shunk, Corps of Engineers, U. S. A., has been published by the War Department.¶ Valuable data showing the application of the deductions of Professor Sadler and others to the case of the channel of the Upper Hudson River have also been published by the Government.** The conclusions reached from the model tests agree fairly well with the results of the tests with full-sized boats, and these should be a guide for engineers in planning new waterways, as well as for naval architects engaged in the design of boats.

A full discussion of this difficult subject would be too extensive; some of the conclusions reached, however, can be given.

The best dimensions for channels, from the standpoint of forming commercial lanes, depend on the form, dimensions, and speeds required for the carriers. These, in turn, depend on the nature of the commercial movement. In the case of natural waterways it may be found that adequate channels cannot be formed, or that the cost of the required improvement would be excessive. In the case of canals, except for the supply of water for the

* *Transactions*, Am. Soc. C. E., Vol. IX (1880), p. 99.

† Report on Barge Canal, State of New York, 1901, p. 599.

‡ "Some Important Phases of Canal Navigation, Illustrated by Recent Experiments in Germany," *Transactions*, Am. Soc. C. E., Vol. XLVII (1902), p. 435.

§ "The Panama Canal," Pt. I, p. 69, Govt. Printing Office, 1905.

‖ The reports of this Board are published in H. R. Doc. 857, 63d Cong., 2d Sess., and H. R. Doc. 108, 67th Cong., 1st Sess.

¶ "Reports on Model Tests of Barges in Waterways of Restricted Dimensions," 1918.

** Appendix G of H. R. Doc. 1160, 63d Cong., 3d Sess.

summit level, cost alone limits the dimensions to be given. In either case if interest on the cost of the improvement plus the cost of its maintenance exceeds, or even equals, the demonstrable saving in transportation costs, the improvement should not be made.

So many varying conditions exist as to the width, depth, and constitution of the bed of navigable streams, and to a certain extent also of artificial waterways, that the formulas for channel and carrier dimensions and forms are empirical. In general, it may be said that in considering a project for a waterway to serve a certain volume and class of traffic, the economic type of carrier, its capacity, and required speed of movement should first be determined. Next, the dimensions required for the channel should be fixed. Again, it should be remembered that if the requirements thus determined make the proposed waterway too costly, the project should be abandoned.

Type of Carrier.—In selecting the type of carrier some of the governing principles are, as follows: As each carrier is only a trussed body, the general principle applies that, other things being equal, the deeper the truss the stronger it will be for the same weight of material. Increase of draft tends toward economy of construction.

When rapidity of transit is important, single, self-propelled carriers, with moulded hull, are best. When it is desirable to move large volumes of freight at a minimum cost, towed barges are more economical, provided the available channel has sufficient width and the bends have sufficient radius. The towing system also makes it possible to keep the expensive towboat in use while the separate carriers are waiting for loading or discharging. As a result of tests on the Mississippi the Experimental Tow Boat Board reported as follows:* "In going ahead, the resistance per ton of displacement decreases as the number of barges increases, if the formation is kept narrow."

Type of Towboat.—The U. S. Tow Boat Board made many experiments as to the best type of towboat for use on the Mississippi River, and its conclusions and the method of attaining them are available.† For the Mississippi River, the Board recommended for towboats either steel, stern-wheel steamers of 4½-ft. draft, with 1 800-h. p. engines, or 1 800-h. p., steel, screw-tunnel boats of 7-ft. draft.

A discussion on towboats for the Hudson River has also been published by the Government‡ in which it is stated:

"Experience has shown that from 13 to 15 ft. is the most economical draft for towboats of the type used on the lower Hudson and in New York Harbor * * *. Based on the best boats now in service on the Hudson, the following seems to be the limiting indicated horse-powers practicable for various draft tugs:

"Draft.	I. H. P.
6 to 8 ft.	150 to 300
8.5 to 10 ft.	500 to 800
13 to 15 ft.	1 000 to 1 800

With equal indicated horse-powers, the pulling power generally increases with the draft. The smallest tugs in service are operated with a crew of 9 men,

* H. R. Doc. 108, 67th Cong., 1st Sess., p. 13.
† *Loc. cit.*, pp. 10–12, 15.
‡ H. R. Doc. 1160, 63d Cong., 3d Sess., p. 103.

and the largest with a crew of 12 men. For a given total pulling power, however, the coal consumption varies more nearly with the total horse-power required, however distributed between tugs. The cost of wages and repairs is more nearly a function of the number of boats. These figures give some idea of the economy in using deep draft boats.

"It has been found, by experiment and experience, that the standard formulas for resistance to motion in general use in marine architecture do not give reliable results for the channel conditions prevailing on the upper Hudson. The best available guide is believed to be contained in curves deduced by Professor Sadler as a result of his model-tank experiments referred to above and are presented in an article entitled 'The Resistance of Some Merchant Ship Types in Shallow Water', read at the Eighteenth General Meeting of the Society of Naval Architects and Marine Engineers, on November 16 and 17, 1911. Based on his curves for 'full type of cargo boat', the following table [Table 1] of resistances and effective horse-powers were computed for a barge 270 ft. long and 40 ft. beam, drawing 9.5 ft. The effective horse-power is that computed as necessary to exert the towline resistances given."

TABLE 1.—RELATION OF TOWLINE TENSION AND POWER REQUIRED, FOR VARIOUS SPEEDS AND DRAFTS.

SPEED.		TOWLINE TENSION, IN TONS.			EFFECTIVE HORSE-POWER REQUIRED TO OVERCOME TOWLINE TENSION = ½ I. H. P.		
In miles per hour.	In feet per second.	D = 12 ft.	D = 14 ft.	D = 16 ft.	D = 12 ft.	D = 14 ft.	D = 16 ft.
4	5.866	1.35	0.75	0.66	28.8	25.7	23.5
6	8.8	2.1	1.5	1.35	67.5	48.0	43.0
8	11.74	5.25	3.0	2.55	251.4	128.1	108.86

Tows on the Mississippi River are pushed by towboats, steering being accomplished by the towboat rudders and by the propelling wheel or screws. Use is also made of the pressure of the current on the side of the tow. Much of the time, particularly in down-stream movements, the towboat wheel is backing in order to throw the head of the tow around—this operation is termed "flanking". On the Hudson, tows are hauled on a line. This difference in practice causes a marked difference in the class of towboats used.

Type of Barges and Barge Resistances.—The type of hull to be selected for barges is largely dependent on the relative importance of carrying capacity and of water resistance. In still water, at low speeds and in channels of suitable dimensions, the relative importance of the form of hull decreases; but in natural waterways, as the resistance to movement increases very rapidly with an increase of speed, and as the velocity of the boat with reference to the water must be sufficient to afford a suitable rate of progress when moving against a current, the form of the barge becomes of primary importance.

After numerous experiments in the canalized section of the Seine River, M. de Mas, Inspecteur Général des Ponts et Chaussées, proposed* as the best type two barge forms, each 126.7 ft. long by 16.4 ft. wide by 7 ft. deep (length-width ratio = 7.7), having a spoon-shaped bow and stern, but with

* "Rivères à Courant Libre", 1899 Edition.

different rakes, 14.8 ft. long at each end for one, and 8.2 ft. for the other. He states that the resistance offered by the water to the movement of a boat for all speeds greater than 1.7 miles per hour, can be expressed by the formula, $r = (a + b\,t)\ V^{2.25}$, in which, r is the resistance, in kilogrammes; a and b are constants varying with the form of barge; t is the draft, in meters; and V is the speed, in meters per second.

M. de Mas deduced the following constants for barges of varying forms but of similar dimensions, 38.5 by 5 by 1.9 m.:*

Péniche	$a = 21.3$	$b = 123.6$
Flûte	$a = 21.5$	$b = 78.1$
Toue	$a = 14.2$	$b = 52.4$

In comparison, the models he recommends, at a speed of 1.5 m. per sec., offer a resistance only two-thirds as great as the flûtes. The experiments covered only a limited range as to boats, speed, and channel dimensions, and the conclusions are valid only within that range.

After many experiments, the U. S. Tow Boat Board recommended the construction of steel barges for the Mississippi River freight lines on a model quite similar to those recommended by de Mas. This type of barge is 300 ft. long by 48 ft. wide by 10 ft. deep (length-width ratio = 6.2). It has a modified spoon-shaped bow and stern, 30 ft. wide at the top, with a rake 42 ft. long, and has rounded bilges of 5-ft. radius. At 6-ft. draft, its load is 1 660 short tons; at 8-ft. draft, 2 500 short tons; and at 9-ft. draft, 3 000 short tons. In practical operation this form of scow has shown a resistance markedly less than the usual type. The resistance per short ton of displacement developed by a tow of four of these barges, arranged two abreast, at a speed of $5\frac{1}{2}$ miles per hour, was 2.15 lb., while a similarly arranged tow of barges of the usual scow shape, moving at the same speed, developed a resistance of 3.6 lb. per ton of displacement.

The U. S. Tow Boat Board in its discussion of the resistances to motion of barges† shows that the economical speed for up-stream towing is 50% greater than the velocity of the stream; the Board states:

"The resistances to the motion of a boat are classified as frictional and residual, or wave making. The former is dependent upon the extent and character of the wetted surface of the boat and the speed of movement, and the latter upon the form of vessel, the depth of water and the speed."

The Board quotes Froude's formula for the frictional resistance, namely, $R_f = F\,S\,V^{1.83}$, in which R represents the frictional resistance; F, Tideman's constant = 0.0095, which is the frictional resistance, in pounds per square foot at a speed of 1 knot; V, the speed, in knots; and S, the wetted surface, in square feet. The Board also quotes Taylor's formula giving the minimum depth of water required in order that the resistance may not increase with increased speed, namely, depth $= 10\,H \dfrac{V}{\sqrt{L}}$, in which, H = draft, in feet; V = speed, in knots; and L = length of water line, in feet.

* The péniche is a scow with slightly rounded ends and a small rake. The flûte is a scow with ends similar to the usual canal-boat form. The toue is also similar to the canal-boat form, but with a longer rake at the bow.

† H. R. Doc. 857, 63d Cong., 2d Sess., pp. 6–7.

The Board made experiments in a broad waterway to determine resistances, with results shown in Table 2; in a narrow waterway the friction on the banks would enter to a greater extent. It will be noted that the form (or wave-making) resistance is more than half the total and that proportionally the form resistance increases rapidly as the depth of water decreases.

TABLE 2.—COMPARISON OF RESISTANCES TO A SCOW-SHAPED BARGE FOR DIFFERENT DEPTHS OF WATER.

	In deep water.	In 12 ft. of water.	In 9 ft. of water.
Total resistance (by experiment)	2 371	3 250	4 350
Frictional resistance (by formula)	997	997	997
Residual resistance (difference between above)	1 374	2 253	3 353
Portion of resistance due to surface, percentage	42	30.7	22.9
" " " " " form, percentage	58	69.3	77.1

Constructor McEntee states:

"The resistance expressed as a percentage of the displacement increases very quickly with the speed. For the best form of barge* in a canal of moderate size, the speed in miles per hour cannot exceed $0.55 \sqrt{L}$ ($L =$ length of barge, in feet) without increasing the resistance beyond one per cent. of the displacement * * *. The draft of the barge should be about half the depth of water in the canal, but it can be increased to three-fourths of the depth of water without greatly increasing the resistance providing the sectional area ratio (ratio between the area of maximum wetted cross-section of barge and area of cross-section of the canal) is not changed. With a given cross-sectional area of barge, increase in length reduces the resistance per ton at the same speed. For a barge of length seven times its breadth an increase of 30 per cent. in length will reduce the resistance per ton about 10 per cent. to 15 per cent."

The best type of barge for use under given circumstances is so dependent on the condition of the waterway in which it is to be navigated that the state of the channel must be known before a selection can be made.

Channel Dimensions.—This subject has been considered in several of the sessions of the International Congress of Navigation, and in the reports of the Congress are given the practice in Europe and the conclusions of European waterway experts. In studying European practice, however, it should be borne in mind that with their low standard of wages and low cost of living, the time element, which comprises at least 50% of all the movement costs, permits of the use of speeds which in the United States would be too slow to be economical. In Europe, too, the higher cost of movement by rail allows a margin for increased costs by water which is not found in the United States. Except on rivers like the Rhine and Danube, the barges used on European waterways are too small for economical general use in the United States.

In the European canals, revetment of the banks is general, and frequently a berm is constructed at about the lower limit of destructive wave action. In Russia, this point is found to vary from 13½ to 40 in. below the normal water level. In rock cuts, for one-way traffic, the minimum canal width rec-

* Constructor McEntee's experiments were made mainly with models in which the length-beam ratio was 7, the ratio found to be the most advisable in earlier tests.

ommended is 1.4 times the width of the largest boat. In order to allow boats to pass each other, this width should be increased to 2.8 times the width of the largest boat. Another rule, for boats of 300 tons, recommends a least width for the area having the maximum depth, of double the greatest width of the boat plus 16.4 ft., increasing this width by 1.64 ft. for each additional 100 tons in size. The side slopes used vary with the nature of the material of the banks. The depths recommended are from 1.5 to 3.28 ft. below the bottom of the deepest draft boat. When towboats are propelled by a screw, a clearance of 2 ft. 6 in. is recommended. European authorities recommend that the ratio between the cross-section of the canal and the cross-section of the immersed part of the boat should be from 3 to 5.

Experimental Results.—The results of the tests of the U. S. Tow Boat Board with models and barges of various sizes and in channels of various dimensions, are shown by curves published in its reports. Constructor McEntee's tests were made with models and in his "Report on Model Tests of Barges", previously mentioned, the curves published show the results obtained by him. He summarizes his conclusions from these tests as follows:

"1.—The resistance expressed as a percentage of the displacement increases very quickly with the speed. For the best form of barge in a canal of moderate size, the speed in miles per hour cannot exceed 0.55 $\sqrt{L}$ without increasing the resistance beyond 1% of the displacement. In this statement the quantity $\sqrt{L}$ = square root of the barge length in feet. For a 100-ft. barge this limit would be about 5.5 miles per hour.

"2.—After speed the most important element affecting resistance is the ratio of the cross-sectional area of the canal to that of the barge. For a given barge running at a moderate fixed speed as this ratio is increased from 4 to 8, the resistance is reduced as much as 50 per cent.

"3.—The proportions of the canal—that is, the ratio between its width and depth for a given sectional area—is a factor of importance. For canals of varying width-depth ratio, but of sectional area bearing a constant relation to the sectional area of a given barge, the best results are obtained when the width of the canal at the bottom is about two times the depth of the water. On the other hand, if the beam and length of the barge are increased directly as the width of the canal, the draft being maintained at 75 per cent. of the depth of water, the best width-depth ratio for the canal is about 9 to 11.

"4.—For self-propelled barges, or when towboats are used, the deep and narrow canal will have decided advantages resulting from the possibility of using larger and more efficient propellers than could be used in the shallow and wide canals. This conclusion is based upon estimates, and not on the results of the present experiments except in so far as the resistance of the barges is concerned.

"5.—The draft of the barge should be about half the depth of water in the canal, but it can be increased to three-fourths of the depth of water without greatly increasing the resistance, provided the sectional area ratio is not changed.

"6.—With a given cross-sectional area of barge, increase in length reduces the resistance per ton at the same speed. For a barge of length seven times its breadth an increase of 30 per cent. in length will reduce the resistance per ton about 10 to 15 per cent. The length is limited by considerations of strength, handling, locks, etc., and not by questions of resistance."

As a result of the tests, the conclusion may be reached that, in general, when the waterway is to be used by boats of all sizes, the depth should be

25% greater than the maximum draft of the largest boat, and that the ratio of bottom width of channel to depth should be 11; but if the waterway is to be used only by barges of one size and which have the same length-beam ratio, the resistances are least for a ratio of width to depth of channel of 2, which would make the channel too narrow for two-way traffic. Therefore, the minimum width of channel is considered to be 2.5 times the beam of the largest boat using it. From Constructor McEntee's curves it is evident that, when the time element is of primary importance or when by reason of a current speed is essential, the cross-section of the canal must be increased to as great an extent as would be warranted by the commercial value of the waterway. In commenting on Colonel Shunk's discussion previously referred to, the Acting Chief of Engineers states:

"* * * the apparent advantage of increasing the draft to nearly equal the depth of water, on account of the greater tonnage over which time costs will be distributed, may be more than nullified by the greater risks of striking sunken obstructions, or of being delayed by unexpected shoaling of the channel, by the difficulty of steering when the clearance is small and by the reduction of propeller efficiency."

Parenthetically, the writer, because of Colonel Shunk's mistaken assumptions, is compelled to disagree completely with the conclusions reached.

Constructor McEntee's experiments were made with models of different sizes, in channels of various cross-sections. In his "Report on Model Tests of Barges," he gives a number of curves of resistance for different ratios of length to beam of barges, for different ratios of boat and canal cross-sectional areas, and for different speeds. In these curves the ordinates are resistances in pounds per ton of displacement and the abscissas, speeds in terms of $\frac{V_m}{\sqrt{L}}$, in which V_m represents the model speed, in miles per hour, and L, model length, in feet, "for use in extending model resistances to resistance of full sized barges, for the reason that a large part of the total resistance follows the law that it is proportional to the displacement, provided the speeds of the model and the full-size barge are in proportion to the square roots of their length." Under this law, when the resistance of the water, in pounds per ton of displacement, is the same for the barge and the model:

$$V_m : v_m :: \sqrt{L} : \sqrt{l} \therefore V_m = \frac{v_m \sqrt{L}}{\sqrt{l}}$$

in which, V_m, v_m, L, and l are the speed and length of the barge and model, respectively.

Design of Cross-Section of Waterway.—To apply this formula practically let it be assumed that conditions require the use of a barge with a cargo capacity of 2 000 tons at a draft of 7 ft., and that a minimum speed of 5 miles per hour is required. What must be the cross-section of a canal which will permit this speed to be attained with a less resistance per ton of displacement than the 3-lb. draw-bar pull on an engine of a competing freight train?

Let it be assumed also that wharf lengths, canal curvature, lock lengths, etc., will permit the use of a barge similar to the Upper Mississippi River

barges.* This barge at 7-ft. draft has a displacement of 2 575 tons with a load of 2 075 tons.† It is 300 ft. long, 48 ft. beam, and the ratio of barge length to model length is 21.43.

Under the required speed, $\frac{V_m}{\sqrt{L}} = 0.29$. From the relation, $\frac{D}{d} = \lambda^3$, in which, D and d = the displacements of the barge and model, respectively, and λ = the ratio of the lengths of the barge and model; $d = 523.3$ lb. This is slightly less than the displacement of Model 1984^A previously mentioned, at a draft of 6 in. For this value of $\frac{V_m}{\sqrt{L}}$, the model has a resistance of 2 lb. when the sectional area ratio is 7; of 2.6 lb. when the sectional area ratio is 5.6; and of 4 lb. when the sectional area ratio is 4.67.‡

The area of the wetted cross-section of the barge is 48 by 7 = 336 sq. ft. The canal of smallest cross-section to permit with certainty the speed to be developed with a resistance of less than 3 lb. per ton of displacement is evidently that for which the sectional area ratio is 5.6. The canal, therefore, must have a cross-section of 5.6 by 336 = 1 881.6 sq. ft.

Assuming that for local reasons the canal depth should be 3 ft. greater than the barge draft, or 10 ft., the required width becomes 188 ft., or 3.9 times the barge width.

In the discussions of the U. S. Tow Boat Board experiments, important elements which affect the channel widths and the depths required in practice (such as the effect of wind action, erosion of banks, etc.), are not considered.

Squat of Boats.—In fixing the proposed channel depth, the "squat" or increase of draft of a boat when under way, must be taken into account, although it is slight at low speeds. Discussions on the squat of self-propelled boats§ and of barges‖ have been published. For Hudson River traffic, using boats drawing 10 ft. of water and moving at a speed of 10½ miles per hour, the writer concluded that on account of the squat and of the impossibility of keeping the river bed completely free from obstructions, safe clearance cannot be assured with depths of less than 14 ft.

Bends in the Channel.—In rounding a bend, the channel width required for a single boat, or for a tow, is increased considerably beyond the beam of the tow. Herr Gröhe, Inspector of Waterways, Münster, Germany, reported to the VIth Navigation Congress, as follows: "We cannot go below a radius of 500 m. for canals, which allow of a rapid navigation with at most two boats in file." Several formulas have been developed to show this required increase in width. Observations on the movement of boats and tows in rounding bends were made in New York Harbor and in St. Mary's River. Two of the formulas were found to give results closely in accord with the observations,

* H. R. Doc. 108, 67th Cong., 1st Sess., p. 72.

† The form of the barge is quite similar to that of Model 1948^A, shown on Plate 3 of McEntee's "Report on Model Tests of Barges".

‡ "Report on Model Tests of Barges", Plate 22.

§ H. R. Doc. 1160, 63d Cong., 2d Sess., Inclosure G, p. 101.

‖ "Report on Model Tests of Barges", p. 18.

namely, that of M. Van Der Linden, $d = 4\left(r - \sqrt{r^2 - \frac{l^2}{4}}\right)$, in which d = the increase in width, in meters, necessary; r = radius of curve, in meters; and l = the length of the boat, in meters; and one deduced in the First New York District, River and Harbor Improvements, namely, $d = \sqrt{r^2 + l^2} - r$, in which d = increase required, in feet; r = radius of curve, in feet; and l = length of boat or tow, in feet.*

Wind Action.—When boats are exposed to wind action from the side, they assume a more or less oblique position with respect to the axis of the channel, and thus require more channel width. In a published discussion on this subject† the tractive and wind forces acting on the carrier are determined and the direction of the resultant is found. From the calculations, it was concluded that a self-propelled barge, 300 by 42 ft., with draft of 10 ft. and a free-board of 6 ft., moving at a rate of 4 miles per hour, and exposed broadside to a wind having a velocity of 9 miles per hour, will occupy 83 ft. of channel width; and that when, under similar conditions, the carrier is towed on a line, the tow-line will lie at an agle of about 10° 30′ with the axis of the channel.

Erosion of the Banks.—The effect of the wave produced by the movement of a boat through the water must also be considered, especially in restricted channels. The height of this wave is a function of the form and speed of the boat and of the dimensions of the waterway. The *Hendrik Hudson,* moving in the Hudson River near West Point, N. Y., at a rate of 22 miles per hour, produced a swell about 1 ft. high which became inappreciable at a distance of 1 000 ft.; passing Bogart Island near Albany, N. Y., in a channel, 770 ft. wide and 12 ft. deep, the corresponding swell at the shore was 5½ ft. high, from hollow to crest.

When a boat is in motion there must be a constant movement of water, in a direction opposite to that of the boat movement, to fill the space the boat has just left. In restricted channels, even at low speeds, this water must move with a considerable velocity. In effect, the boat is climbing a grade; the water takes a slope down from bow to stern and, with the velocity due to the head created, rushes past the sides and under the bottom of the hull. The smaller the ratio between the sectional areas of the boat and the channel, the less will be the space through which this water must pass and the greater the velocity and consequently the head required to produce the velocity; and the greater the speed of the boat, the greater must be the velocity of flow. Bank erosion is caused by this flow in addition to the effect of the boat waves. If channel depths are to be maintained, this action must be considered and provision for it must be made.

Tows.—The methods used for towing cargo carriers has a marked influence on the type of towboats and carriers as well as on the required channel dimensions. In Europe, towboats are generally propellers and haul their tows by one or more lines. On the Rhine, a separate line from the towboat to each

* See H. R. Doc. 391, 62d Cong., 2d Sess., p. 160.

† H. R. Doc. 1387, 63d Cong., 3d Sess., p. 50.

boat of the fleet is used, and each boat has its rudder and helmsman. This enables the boats to take a broad or a narrow formation, to suit channel conditions. In the eastern part of the United States, propeller towboats are used, hauling with a tow-line. In special cases, with one barge only, the tow-boat is fastened to the side of the barge; with two, between the barges. In the barge fleets of the Hudson River, bound to or from the Barge Canal, as many as sixty boats are made into a compact fleet, four or five abreast, and hauled by one or two tugs.

Boats that use the canal generally have full moulded forms with rudders. On the canal they are towed in small fleets of two to four. Boats that use the Champlain Canal and the crooked channel of the Narrows of Lake Champlain frequently are towed in a single line, and attached to each other by bridles which can be manipulated by winches, for steering the individual boats. On the Mississippi River, the barges are generally of scow form, without rudders, and are made into fleets by fastening the barges together as rigidly as possible with lines, thus forming a single floating unit which is pushed by the towboat. Stern-wheel steamers are preferred for this work, although tunnel-boat propellers are sometimes used. There is a record of a single tow, 258 ft. wide and 648 ft. long, made up of 27 coal boats, 8 coal barges, and 2 fuel boats, carrying about 25 000 long tons of coal. Such a tow is comparatively unmanageable and can be handled only down stream and in a broad channel. The U. S. Tow Boat Board found that the best formation for a tow of six barges was two abreast and three deep, the resistance per ton of displacement, in still water, with a speed of 5.25 miles per hour, for this formation being 3.13 lb., while in a formation three abreast and two deep the resistance was 3.54 lb. per ton. It is evident that, to permit such a tow to operate, channels must be broad and fairly straight.

Costs of Transportation.—These costs must vary with the conditions. If there is a line of water carriers running on a fixed schedule, with full cargoes offered in each direction, the cost of operation should be a minimum and a manufacturer located on the waterway can compare directly waterway and rail charges and advantages. If, however, the waterway is designed to serve manufacturers who must own and operate their own floating plants and, therefore, use the waterway intermittently, the economy of water carriage is decreased. For the towboats as well as for the barges, the cost in wages and in interest on the investment must be added for "lay" days and that cost plus the actual cost of operation must be apportioned to the total tonnage carried. Should the required run be longer than the working day, there will be extra charges for overtime, if not indeed for wages of a duplicate crew.

In studying a project for a waterway all the foregoing elements of cost must be given due weight. Conditions which control the cost of the construction and maintenance of channels in natural or artificial waterways differ so widely that no general equation of costs can be given, just as at present there is no reliable general formula for the resistance of boats. When the variable factors are known, however, deductions which are approximately correct can and should be made.

In comparing the economic transportation value of an existing or proposed waterway with that of a railway, the special conditions pertaining to each must be considered, bearing in mind that the addition of one or more trains per day on a railroad already in operation causes an insignificant increase in the general overhead operating costs. Data as to railways are widely scattered.*

C. McD. Townsend, Col., U. S. A. (*Retired*), M. Am. Soc. C. E., gives† an enlightening discussion on this subject and makes direct comparisons between rail and water transportation. In it he makes the statement:

"The increase in size of the freight car has enabled the locomotive to haul 200 tons of freight at the rate of 5 miles per hour with the same application of force that was required on the canal boat also carrying 200 tons and moving in the contracted waters of a canal at the rate of 2 miles per hour".

In this case, however, he compares efficient modern railway equipment with out-of-date waterway equipment. From the table compiled by Mr. Lavis,‡ it is found that the hauling capacity of a Consolidation engine (tractive power, 60 900 lb. to 62 500 lb.), under ordinary conditions, on a road with grades of 0.5%, moving at 20 miles per hour, is 2 160 tons, equivalent to a train of thirty 50-ton cars, carrying 1 500 tons of freight. Mr. Lavis also states:§

"For trains of loaded 50-ton cars, all of the same type, the resistance at ordinary freight train speeds is often as low as 3 lb. per ton, whereas trains of miscellaneous types and sizes, including some cars as light as 25 to 30 tons and some empties, will have a resistance as high as 6 to 9 lb. per ton, the usual average, however, being about 6 lb. per ton."

As stated previously, the resistance offered by a fleet of six ordinary Mississippi River barges, weighing 141 tons each, and with a capacity of 717 tons each at 6-ft. draft, moving in still water at a rate of 5½ miles per hour, was 3.13 lb. per ton of displacement. Two barges of improved type, moving at the same speed developed a resistance of only 1.87 lb. per ton; each of these barges weighs 500 tons, and at a draft of 9 ft. carries 3 000 tons.

A rough comparison between the freight-carrying capacity of a double-track railroad and that of an adequate sea-level canal may be made. The usual minimum headway allowed between trains is ½ hour. If this headway is taken as 20 min., 72 trains per day can be run each way. If forty 50-ton cars are allowed per train, instead of thirty as previously mentioned, the carrying capacity per train will be 2 000 tons, and the total freight-carrying capacity per day for a single track will be 144 000 tons. A safe interval for boats moving at a rate of 5 miles per hour on an adequate waterway is 5 min. Under these conditions, 288 boats could arrive at a given terminal per day. If these boats are 3 000-ton carriers (the capacity of the type of barges used on the Upper Mississippi River at a draft of 9 ft.), this would make the one-way

* Much information is available in "Railway Estimates", by F. Lavis, M. Am. Soc. C. E., 1917; further information may be found in "The Maximum Weights of Slow Freight Trains", by C. S. Bissell, M. Am. Soc. C. E., and the discussion thereon, *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 303, and in "The Analysis of Cost of Freight Service, Grand Trunk Railway Company of Canada", by J. P. Newell, M. Am. Soc. C. E., and the discussion thereon, *Transactions*, Am. Soc. C. E. Vol. LXXXVI (1923), p. 351.

† "Rivers and Harbors", Chapter XV.

‡ "Railway Estimates", p. 445.

§ *Loc. cit.*, p. 444.

freight-carrying capacity of the canal 864 000 tons per day, or six times that of the railway.

The Future of Railway and Waterway Transportation.—The writer is convinced of the advisability of making a fuller use of the natural advantages possessed by the United States in its waterways, as a means of providing the economy in transportation so much needed for the nation's life. He is equally convinced, however, that the opening of inadequate waterways means only a waste of public funds. He hopes that this paper may serve to invite attention to facts which seem to have been overlooked by the radical enthusiasts among both the railway and waterway advocates, and that a further discussion of the subject may form a step toward that most desirable end—the substitution of co-operation for destructive competition between waterway and railway carriers.

THE HYDRAULIC DESIGN OF THE SHAFT SPILLWAY FOR THE DAVIS BRIDGE DAM, AND HYDRAULIC TESTS ON WORKING MODELS

Discussion*

By Messrs. F. W. Scheidenhelm and H. K. Barrows

F. W. Scheidenhelm,† M. Am. Soc. C. E.—This paper is one which, however much it is valued at the time of its presentation, will be most deeply appreciated by engineers who later refer to it in connection with analogous problems. It contains a concise, yet clear, explanation of the theory, design and experimentation pertaining to the subject of shaft spillways. Moreover, it possesses the exceptional merit of not leaving the reader in doubt as to pertinent assumptions or conditions affecting this phase or that.

Several requisites are involved in such a contribution as that of the author. As a matter of course, there is first the detailed design, supplemented in the present instance by experimental study. Generally speaking, it is only for an important project that such extensive and expensive theoretical and experimental investigation is involved or can be afforded. Consequently it is necessary that the appropriate executives consent to the publication of the results of the work. In the present instance the profession should duly appreciate the willingness of the New England Power Company and the J. G. White Engineering Corporation that this paper should be prepared and the results made public by a responsible engineer as a part of his due contribution to engineering knowledge. Ordinarily at the conclusion of the design the office records are not in shape for publication. Granting the requisite ability, there is necessary the willingness on the part of an author to devote a great deal of time and energy to preparation of the material, for, when once the engineering work is done, he has a strong temptation to place the subject-matter on a shelf, inaccessible to fellow engineers.

The paper sets forth the evolution of this particular design and its experimental demonstration and adjustment in such detail as to create a longing for more, namely, for a demonstration in full-size operation. This is something which, presumably, only Nature can afford; but inasmuch as it appears probable that this shaft spillway will be required from time to time to take care of floods which are not dwarfed into insignificance by the capacity of the reservoir to absorb flood peaks, it is to be hoped that the results of such occurrences will be accurately observed and similarly published. This hope finds the greater basis in the fact that this spillway serves one of a number of hydro-electric power plants along a river which is being developed inten-

* Discussion of the paper by Ford Kurtz, M. Am. Soc. C. E., continued from May, 1924, *Proceedings*.

† Cons. Engr. (Mead & Scheidenhelm), New York, N. Y.

sively. One may therefore anticipate close hydraulic supervision by a capable technical staff that will record both the pertinent reservoir levels and the corresponding discharges through the shaft spillway.

The determination of appropriate dimensions, of frictional and flow factors for homologous models to be used in experimentation, is a complicated and important matter. One of the incidental features of the paper, but one which the speaker rates as of considerable importance, is the exposition covering the design and choice of factors for the models which were tested.

The willingness to subject surfaces of concrete to contact with water flowing at so high a velocity as 68 ft. per sec. (average), requires considerable courage. There are few, if any, precedents by way of structures intended to endure similar velocities. The speaker would be surprised if, on the occurrence of velocities approximating the anticipated maximum, any considerable roughness or spalling in the concrete did not result in material local erosion, extending perhaps through the lining or even into the rock backing. Fortunately the conditions are such that no extensive harm could result and the waterway, having performed its duty of taking care of a critical flood, could be patched or otherwise repaired. Performance in this respect for velocities beginning as low as 20 ft. per sec., constitutes one of the important features which should be made public from time to time in order to complete the story which the author has so admirably begun—and in fact has carried well along toward its concluding chapters.

The paper as a whole leaves the speaker with the impression that the design is in exceptional measure based on rational analysis instead of mere judgment.

H. K. Barrows,* M. Am. Soc. C. E. (by letter).†—There are many interesting features in the design and construction of the Davis Bridge Spillway and Dam and the author is certainly to be commended for his comprehensive and detailed presentation of the methods used in experimentation and in the design of the shaft spillway. In adding to the discussion of this paper, the writer will present the results of further experimental work on the model spillway made at the Massachusetts Institute of Technology in 1923.

Necessary Capacity of Spillway.—The record of flow of the Deerfield River at Charlemont, Mass., from 1913 to date affords a fairly good basis for an estimate of the flood flow to be expected at Whitingham, Vt.

For the ten years of record at Charlemont (for an area of 362 sq. miles, less 30 sq. miles at Somerset Reservoir, or 332 sq. miles net), the "average yearly flood" is 10 900 sec-ft., or a flood coefficient of $C = 105$, in the formula, $Q = C\,A^{0.8}$, as used by Weston E. Fuller, M. Am. Soc. C. E.‡ Applying this coefficient to the area above Whitingham (184 sq. miles, including for safety the 30 sq. miles at Somerset Reservoir) gives an average yearly flood to be expected at Whitingham of about 6 800 sec-ft.

Using $T = 1\,000$ years, there results a maximum daily flow of 23 100 sec-ft., with a crest flow of 33 000 sec-ft. (or 3.4 times the average flood); using

* Prof. of Hydr. Eng., Mass. Inst. Tech.; Cons. Engr., Boston, Mass.

† Received by the Secretary, May 22, 1924.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564 *et seq.*

$T = 10\,000$ years these quantities become 28 600 and 40 500 sec-ft., respectively (or 4.2 times the average flood).

At the vicinity of the spillway level of the Davis Bridge Dam (Elevation 1 386), the water area of the reservoir is about 2 200 acres. Assuming a parabolic distribution of flow during the flood day, and starting with water at spillway level (Elevation 1 386.0), detailed studies give the hydraulic data shown in Table 10.

TABLE 10.—HYDRAULIC DATA AT DAVIS BRIDGE DAM FOR 1 000-YEAR AND 10 000-YEAR FLOODS.

Item.	A. 1 000-year flood.	B. 10 000-year flood.
Flow into reservoir for 24 hours of maximum flood day, in second-feet	23 100	28 600
Maximum crest flow into reservoir during flood day, in second-feet	33 000	40 500
Maximum elevation of water surface on spillway, in feet	1 393.7	1 395.0
Corresponding head on spillway, in feet	7.7	9.0
Maximum discharge over spillway, in second-feet	27 400	34 400

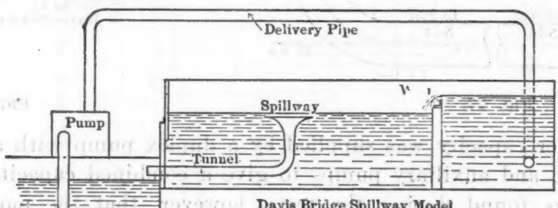
In the foregoing studies, a value of $C = 2.85$ in the formula, $Q = C L H^{\frac{3}{2}}$, for the spillway was used. As a matter of fact the capacity of the tunnel limits the possible discharge of the spillway to about 30 000 sec-ft. Due to this limitation the elevation reached by the water surface at the spillway during the 10 000-year flood would have been increased about 0.6 ft., or to about 9.6 ft. Even with the foregoing severe assumption as to possible flood, as the top of the earth dam is at Elevation 1 400, there would still be a free-board of about 4.4 ft.

The highest observed flow at Charlemont was about 45 000 sec-ft. at about 11 P. M. on July 8, 1915. For the 24 hours from noon, July 8, to noon, July 9, the discharge averaged 20 500 sec-ft. For the two days, July 8 and 9, the rainfall at Somerset totaled about 5 in. (following several days of rainfall subsequent to June 30, which amounted to about 3 in.). Assuming the same flow per unit of area at Davis Bridge as at Charlemont (namely, 136 sec-ft. at the crest and 62 sec-ft. as the average for 24 hours from each of the 332 sq. miles below Somerset Reservoir, the latter not contributing to this flood), the 1915 flood would have given a crest discharge at Davis Bridge of about 21 000 sec-ft. and an average discharge for 24 hours of about 9 500 sec-ft. The foregoing 24-hour discharge for the 1 000-year flood is, therefore, about 2.5 times that of the 1915 flood; the assumed 10 000-year flood is about 3 times that of 1915.

Experiments on Model Spillway at Massachusetts Institute of Technology.—Through the courtesy of the engineers of the Power Construction Company, the model spillway tested at Whitingham and at the Worcester Polytechnic Institute, was shipped to the Hydraulic Laboratory of the Massachusetts Institute of Technology late in 1922, so that it was possible to make some

experiments* on the capacity of this spillway and tunnel under conditions differing somewhat from those for either of the previous tests.

The general arrangement of the spillway and testing apparatus is shown in sectional elevation on Fig. 26. The model was set in the 10-ft. canal in the basement of the Hydraulic Laboratory, about 22 ft. up stream from the end of this canal, where ordinarily water passes over a 10-ft. weir. This weir, however, in these tests was tightly closed by a bulkhead in order to maintain the water level in the canal at any desired height with reference to the spillway model. About 29 ft. up stream from the model, a new bulkhead was constructed in which was placed a 4-ft. standard weir with end contractions, the weir crest being formed by the beveled edge of a 6-in. steel angle and the end contractions by beveled steel plates. Readings of head on the weir were made by a hook-gauge in a stilling-box placed about 6½ ft. up stream from the weir, at the side of the canal.



Davis Bridge Spillway Model
Tests at Mass. Inst of Tech.–1923
Arrangement of Apparatus

FIG. 26.

A wooden deck was built in imitation of the bench in the cliff which partly surrounds the full-sized spillway. The approach was shaped to the spiral curve adopted for the design, using sheets of galvanized iron. At Davis Bridge there will be a transition curve of 30-ft. radius rising from this bench to the lip of the spillway. In the model this curve was built of cement mortar moulded into place by a wooden templet of 10-in. radius.

For the model tunnel ordinary stovepipe 7½ in. in diameter was used. Piezometers were placed in the tunnel at three points (Nos. 4, 5, and 6, Fig. 27) to enable pressure readings and loss of head determinations to be made under various conditions of flow. These connections were arranged with gauges consisting of glass U-tubes partly filled with water. Piezometer connections were also made at Points Nos. 1, 2, and 3 (Fig. 27) in the model spillway—the same points that were used for this purpose in the previous sets of experiments.

The model was mounted on a wooden frame and connected with the pipe tunnel by a special fitting. The joint between the model and this fitting was caulked with oakum, plastered with cement, and held securely in place by eight bolts passing through the flange of the model and the planks on which it set. For the pipe ordinary slip-joints were used, covered with elastic cement, and held in place by galvanized iron sleeves tightened with bolts, all seams in

* These later experiments were made by Messrs. E. S. Birkenwald and F. R. Morgan as thesis work for the Master's Degree and were performed in the spring of 1923 under the direction of the writer.

the pipe being closed with elastic cement or solder. The pipe was supported by small wooden horses and weighted down by a cast-iron pipe slung beneath it. The model was weighted down by about 600 lb. of paving material piled on the supporting frame, and anchored against horizontal movement by timbers spiked across the canal.

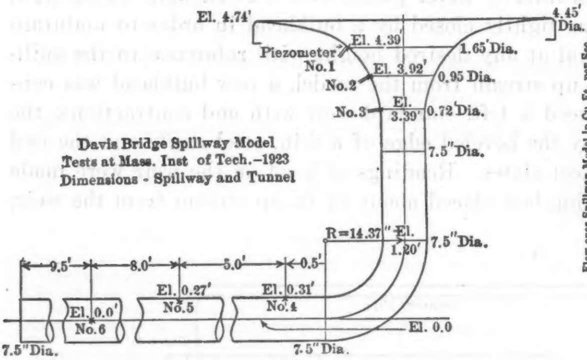


FIG. 27.

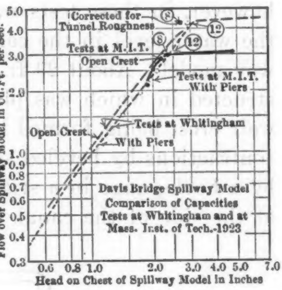


FIG. 28.

Ample water capacity was supplied by a duplex pump with a capacity of about 3 sec-ft. and auxiliary pumps to give a combined capacity of about 6 sec-ft. It was found during the tests, however, that the model could be drowned out by the larger pump alone. Water was delivered at the bottom of the canal about 25 ft. above the weir, giving a stilling-bay approximately 7½ ft. deep, 10 ft. wide, and 25 ft. long, and resulting in a calm and smooth water surface for hook-gauge readings. Runs were made both with and without the flash-board piers, during which measurements of head on the model, pressures at the various piezometers, and discharges were obtained.

There was a slight leakage around the ends of the new bulkhead containing the weir. Careful measurements in several tests showed this to be about 0.04 sec-ft., which amount was used as a correction factor.

Summary of Results.—In Table 11 is given a summary of the more important observed data for those experimental series which are the most typical and representative. Each series is based on three or more individual sets of readings.

Capacity Curves.—On Fig. 28 are given the capacity curves resulting from these experiments plotted with reference to the head on the model and, for comparison, similar curves obtained in the experiments at Davis Bridge, as shown on Fig. 17.* As will be noted, the limit in capacity of the model tunnel in these experiments was reached with a lower discharge than at Davis Bridge, due to the greater coefficient of roughness for the tunnel.

This limit in capacity or "drowning out" of the model spillway occurred at a discharge of substantially 3.0 sec-ft., for which the head on the model spillway crest was about 2.25 in. for the open-crest arrangement and about 2.35 in. for the crest arrangement with piers. Below this critical point, in the

* *Proceedings,* Am. Soc. C. E. December, 1923, p. 1995.

TABLE 11.—SUMMARY OF EXPERIMENTS, DAVIS BRIDGE MODEL SPILLWAY, MASSACHUSETTS INSTITUTE OF TECHNOLOGY, 1923.

Date.	Series.	Pressure Head, in Feet, at Piezometers Referred to Center Line of Tunnel as Datum.					Discharge, in second-feet.	Head on Crest of Model Spillway.		Computed value of C, for spillway model *.	Remarks.
		No. 2.	No. 3.	No. 4.	No. 5.	No. 6.		Inches.	Feet.		
April 28, 1923, (open crest.).	5	3.95	3.44	0.370	0.345	0.091	2.05	1.80	0.150	2.54	....
	6	4.29	3.94	1.162	1.020	0.362	2.65	2.03	0.168	2.74	....
	7	4.31	3.95	1.396	1.074	0.460	2.87	2.16	0.179	2.70	Tunnel nearly full.
	8	4.36	3.97	1.416	1.188	0.560	2.99	2.34	0.195	2.48	Tunnel completely filled.
	9	4.40	3.99	1.409	1.156	0.695	3.07	4.89	0.406	....	....
May 1, 1923, (with flash-board piers.).............	1	3.90	3.36	0.320	0.278	0.008	1.53	1.43	0.119	2.95	....
	8	3.91	3.37	0.320	0.292	0.016	1.78	1.60	0.133	2.93	....
	9	3.92	3.38	0.351	0.312	0.042	1.97	1.73	0.144	2.87	....
	10	3.895	3.37	0.454	0.409	0.217	2.23	1.87	0.156	2.89	....
	11	3.895	3.39	0.767	1.033	0.400	2.68	2.16	0.179	2.85	....
	12	4.25	3.85	1.427	1.155	0.581	3.02	2.45	0.205	2.59	Tunnel completely filled.
May 3, 1923, (with alternate piers removed.)...........	1	3.90	3.36	0.587	0.955	0.376	2.48	1.97	0.164	2.84	....

* Using length of crest of 13.96 ft. for open crest, 12.55 ft. for crest with piers, and 13.25 for crest with alternate piers removed.

tests in which piers were used, the discharge curves are nearly the same for both sets of experiments; for the open-crest type, the experiments at Whitingham show a little greater spillway capacity than those at the Massachusetts Institute of Technology.

Hydraulic Gradient in Model Tunnel.—Pressure readings for piezometers in the tunnel (Nos. 4 to 6 in Table 11) were rather variable and uncertain for a discharge of less than 2.5 sec-ft. A study of the gauge readings and a plot of the hydraulic gradients between Piezometer No. 4 and the end of the model tunnel (see Fig. 27) resulted as shown in Table 12.

TABLE 12.—CONSTANTS FOR TUNNEL DISCHARGE.

Discharge, in second-feet.	Hydraulic gradient in tunnel.	f, in $\frac{f\,l}{d}\,\frac{v^2}{2\,g}$.	C, in $C\sqrt{R\,S}$
2.5	0.045	0.267	98
3.0	0.062	0.264	99

Effect of Model Tunnel on Spillway Capacity.—The model tunnel was $7\frac{1}{2}$ in. in diameter, its hydraulic radius 0.156 ft., and its length 26.3 ft., differing slightly from the exact relative dimensions $\left(\frac{1}{36}\right)$ of the full-sized tunnel which would have been, hydraulic radius, 0.149 ft., and length, 26.6 ft. On the other hand, the difference in elevation between the spillway crest and the center of the tunnel at the exit was 4.74 ft. for the model instead of 4.78 ft. as given on Fig. 2,* showing a slight deficiency of discharge.

For the model tunnel, as noted by the author,† $\frac{l}{C^2}$ should be $\frac{1}{36} \times \frac{957}{136.5^2} = 0.001425$. The value of C^2, if $l = 26.3$ (as at the Massachusetts Institute of Technology) would be 18 400, and C should have been 136, whereas the test value was 99. As the discharge would vary approximately directly as C, the maximum discharge through the model tunnel would have been $3.0 \times \frac{136}{99} = 4.1$ sec-ft., if the model tunnel had been of a length and a degree of smoothness comparable to that assumed for the full-size tunnel.

This greater discharge also would have increased the head on the model spillway to the following amounts:

(*a*).—Open crest:

$$4.1 = 2.70 \times 13.96\,H^{\frac{3}{2}}$$
$$H = 0.222 \text{ ft.} = 2.66 \text{ in.}$$

(*b*).—With piers:

$$4.1 = 2.85 \times 12.55 \times H^{\frac{3}{2}}$$
$$H = 0.236 \text{ ft.} = 2.84 \text{ in.}$$

* *Proceedings*, Am. Soc. C. E., December, 1923, p. 1955.
† *Loc. cit.*, p. 1973.

The computed values of discharge and head on the spillway for Series 8 and 12 are shown on Fig. 28, and accord reasonably well with those of the previous experiments at Whitingham, in which the tunnel was made as smooth as possible, with a length of 17 ft. from the bend to the end of the tunnel (as compared with 23 ft. at the Massachusetts Institute of Technology), with an assumed coefficient of roughness, n, of 0.009 (which was not verified, however, by the piezometer readings).

The corresponding capacity of the full-sized spillway and tunnel, based on the Massachusetts Institute of Technology experiments, is, therefore, approximately as follows:

$$Q = 4.1 \times \frac{398}{0.307} \times \sqrt{\frac{5.36}{0.156}} = 31\,400 \text{ cu. ft. per sec.}$$

The head reached on the spillway crest would be:

(*a*).—Open crest:

$$2.75 \times \frac{36}{12} = 8.25 \text{ ft.}$$

(*b*).—With piers:

$$2.87 \times \frac{36}{12} = 8.61 \text{ ft.}$$

By reference to the author's Fig. 25*, it will be seen that according to his conclusions the ultimate capacity of the spillway tunnel (with piers) was reached at a discharge of about 33 500 sec-ft., with a head on the spillway of about 9.3 ft., as compared to 31 400 sec-ft. and 8.6 ft. based on the Massachusetts Institute of Technology experiments, indicating a slightly smaller tunnel capacity for the latter.

Based on the results of the Massachusetts Institute of Technology experiments alone, for the spillway with piers, the limitation in discharge capacity of the spillway and tunnel of 31 400 sec-ft. would result in a maximum elevation of water surface at the spillway for the 10 000-year flood of about 1 395.3, instead of 1 395.0 based on the previous experimental work. Considering the different methods used in these experiments, however, this is a fairly good agreement.

The model spillway is again at the Hydraulic Laboratory of the Institute, and it is planned to experiment further with it—particularly to make a more extensive study of the losses of head between different sections of the spillway and tunnel.

Use of Tunnel Type of Spillway.—The Davis Bridge spillway is a novel type of construction, the use of which evidently depends on:

1.—A sufficient head at the dam to permit the use of a tunnel section of reasonable size and cost.

2.—A limit in the size of the drainage area and, therefore, of the necessary flood capacity to be handled, due to the size of tunnel section required. Even here, however, the same difficulty would apply in a measure with other types of spillway.

* *Proceedings*, Am. Soc. C. E., December, 1923, p. 2005.

3.—A large water area for the reservoir such that flood peaks may be absorbed by storage above spillway level, thus greatly lessening the maximum capacity of spillway required and reducing the flow to be taken by the spillway to practically an average daily (24-hour) flow instead of, perhaps, that for the maximum hour of the day.

Another field of use for this type of spillway is in the case of arch dams, where it may be impossible to provide a spillway outlet auxiliary to the dam and where the section of the arch dam is not adapted to use as a spillway. It is likely, too, that the general conditions favoring the use of an arch dam, in general, would favor the use of the tunnel spillway.

The behavior of the Davis Bridge spillway and tunnel under conditions of flood flow will be awaited with interest and should furnish valuable data in a field of hydraulics about which information is now very meager. The loss of head at the bend between the vertical shaft and the tunnel is a particularly uncertain element entering into the estimation of the tunnel capacity, both on account of the size of the cross-section and the high velocities, and it is especially to be hoped that this may be determined by measurement.

HIGHWAY RESEARCH IN ILLINOIS

Discussion*

By H. F. Clemmer, Assoc. M. Am. Soc. C. E.

H. F. Clemmer,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author is to be commended on the remarkably clear manner in which he has reviewed the mass of data that has been assembled in the several investigations conducted by the Illinois Division of Highways. The very practical interpretation of the results of the Bates Road tests has effected considerable saving in highway construction costs throughout the country.

The following is offered, not as a discussion of the material contained in Mr. Older's paper, but rather as additional information concerning a few of the investigations described.

Impact.—The author states that at the time of this investigation no suitable instrument could be found for determining the fiber deformation in the upper face of a concrete slab under moving load. Since then a strain gauge has been developed which may be able to measure the stress developed under static loads but which is not presumably reliable under moving loads, due to the overthrow, from inertia, of the recording arm.

It is an ascertained fact that when a concrete beam is in the form of a cantilever, its deflection varies directly as the load and therefore is a measure of the strength. This deflection under static load has been shown by Mr. Older to have a curve of the same radius as that under impact loading and is without doubt a true measure of the effect of impact loading.

Warping Due to Temperature Changes.—The use of the longitudinal center joint to relieve the warping action has unquestionably proven satisfactory, reducing it by one-half. It has not been considered that this warping action caused definite stress. However, recent investigation in the laboratory indicates rather conclusively that temperature differences in concrete do create stress. Considerable further scientific research will be required to determine the effect and magnitude of this stress, if such a stress is created. The Illinois Division of Highways is at present conducting laboratory investigations on the effect of temperature changes on concrete but sufficient data have not been assembled to give a conclusive statement.

Longitudinal Bar Along Edge of the Slab.—Recent tests made to determine the value of the ¾-in. longitudinal bar show that, in the case of temperature cracks, 33 to 46% of the load is carried from one corner to the adjacent corner while at construction joints only 6 to 8% is likewise transferred. During low

* This discussion (of the paper by Clifford Older, M. Am. Soc. C. E., presented at the meeting of the Highway Division, January 17, 1924, and published in *Proceedings* for February, 1924), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, April 28, 1924.

temperature, natural cracks do not open to the same extent as construction joints. From this, it would seem advisable to increase the resistance to shearing action at construction joints.

Fatigue.—The term "fatigue" is accurate, as concrete beams, under repeated load and subsequent to failure, show an increase in deflection and a lack of complete recovery with each load application—a "tiring out". It is interesting to note in this study that the repeated application of a load less than 50% of the modulus of rupture increases the strength of the concrete at the stressed section. In other words, maximum traffic does not injure or wear out a pavement provided the loads are within the so-called endurance limit of the concrete.

Sub-Grade Soil Investigation.—The author's statement, that Illinois soils having a moisture content which may be considered normal for the summer months resist further saturation to a marked degree, has been definitely checked. It has been further established that the bearing value of a soil varies with its moisture content. Several experiments are now being carried on by the Illinois Division of Highways to determine the method and extent of treatment necessary to obtain the optimum moisture for a soil. It has been necessary to develop tests such as mechanical analysis, slaking value and constancy of volume. Considerable data have been obtained on the bearing power of soils having various characteristics and it is believed that means may be found for the treatment of a subgrade so as properly to proportion the elements of the soil—sand, clay and gravel—to control the moisture content and thereby to create a definite bearing power for that soil.

The prime development from "Highway Research in Illinois" that has been checked and proven practical for use is the formula, $d = \frac{\sqrt{3W}}{S}$. Concrete roads designed under this formula and on which traffic loads are properly controlled will last indefinitely.

THE SECONDARY EFFECT OF CERTAIN IMPORTANT RIVER BRIDGES ON LOCAL TRANSIT CONDITIONS

Discussion*

By Messrs. Charles Evan Fowler, M. W. Weir, Herman H. Smith, D. B. Steinman, H. M. Lewis, and T. Kennard Thomson.

Charles Evan Fowler,† M. Am. Soc. C. E.—The author has presented only that feature of a traffic bridge which is of importance to those engaged in rapid transit, namely, its use for passengers; the vehicular traffic, which is of paramount interest to the builder of a toll bridge, has scarcely been mentioned. The income from passengers crossing a toll bridge will represent only about 5% of the total revenue of a bridge some distance from a city; it may amount to 40% for an inter-city structure that carries rapid transit. The major part of this income must come from automobiles, auto trucks, and auto-truck freight. Such traffic was not available in sufficient volume even a few years ago to make possible the financing of the many much needed bridges.

The speaker's detailed study within the last few years of ten or twelve large projects of this type throughout the country, and his close observation of fifty or more toll bridges now in operation, have convinced him that except in extreme cases the passenger traffic from the income standpoint is either negligible or must be subordinated in the design to vehicular traffic. Moreover, automobiles will use a bridge to the exclusion of a ferry, when the bridge entrance is within 2 miles of the ferry crossing, and in case the traffic is of average density; but if the traffic is light, the distance may be much more than 2 miles and still the bridge will be used in preference. This is due to the fact that an automobile needs on an average at least 15 min. merely to get on to the boat, but during rush hours, and on Sundays and holidays, this time may be from 1 hour to several hours.

The bridge over the Columbia River between Portland, Ore., and Vancouver, Wash., may be taken as an example of the class of bridge situated some distance from a city. This crossing was served by one small ferry until February, 1917, when a bridge was completed at a cost of considerably less than $2 000 000. The ferry simply fell into disuse; and the traffic has increased to such an extent that the net earnings of the bridge will exceed its cost in about six years. The curves established for passenger and automobile traffic over the Philadelphia-Camden Bridge, as shown in the report of the Delaware River Bridge Commission, indicate a probable traffic in 1927 of more than 60 000 000 passenger crossings, and more than 4 000 000 automobile crossings.

* This discussion (of the paper by John A. Miller, Jr., Assoc. M. Am. Soc. C. E., presented at the meeting of May 7, 1924, and published in *Proceedings* for April, 1924), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Civ. Engr., New York, N. Y.

Due to the probable increase in population and in the number of automobiles, the speaker would place this latter expectation at 6 000 000. Also, there will be, in all probability, upward of 1 500 000 crossings of auto trucks, carrying about 6 000 000 tons of freight. If this structure should be operated as a toll bridge, the income for the first year would surely exceed $5 000 000.

It is often the case with engineers and generally the case with public officials that they do not fully grasp the trend of modern vehicular traffic. There were about 15 000 000 gasoline vehicles in use in the United States in 1923; with more than 4 000 000 machines being manufactured each year, the number may be expected to reach more than 25 000 000 by 1927, even after deducting those scrapped and otherwise destroyed. It is fair to assume that each summer will find 60% of these automobiles on tour, or about 15 000 000 by the summer of 1927 to complicate the bridge-traffic problem. The increase in the number of automobiles in large cities is much more rapid—about 30 to 40% per annum. The increase due to this summer traffic each year will be about 100% above the normal local annual traffic, and about 50% above the average. This, however, does not represent fully the problem for those who are planning large bridges, as the daily peak load each week will be more than 50% greater than the average daily traffic, and the hourly peak loads, which are not absolutely prohibitive now, will become so in 3 or 4 years. By "prohibitive" is meant that no bridge can be made to pay if constructed to provide for anything approximating the peak loads that will surely come within 2 or 3 years.

The speaker's studies for a long-span bridge across the Detroit River between Detroit, Mich., and Windsor, Ont., Canada, were based on the traffic of the trunk highways and of the city streets, but particularly on extended tallies of the present ferry traffic. This would indicate not less than 15 passenger crossings per capita per annum, as compared with 30 between Philadelphia and Camden, and 50 between San Francisco and Oakland, Calif. The population for Detroit, Windsor, and their environs was estimated as being 1 650 000; for Philadelphia, Camden, etc., as 2 000 000; and for San Francisco, Oakland, etc., as 1 000 000. On the basis given, and in consideration of other data gathered, the minimum traffic for the Detroit River Bridge for 1927 would be as follows:

Passengers	25 000 000
Automobiles	3 000 000
Auto trucks	700 000
Auto-truck freight, in tons	3 000 000

The maximum traffic would be:

Passengers	50 000 000
Automobiles	5 700 000
Auto trucks	2 200 000
Auto-truck freight, in tons	8 800 000

The corresponding gross income would range from a minimum of about $2 500 000 to a maximum of more than $6 000 000, the larger estimate being tenable on account of the very large traffic to and from the 200 Canadian branches of American manufacturing concerns.

The Detroit Bridge is also planned to carry two rapid transit tracks, but in the speaker's opinion all such traffic for New York should be carried by tunnels with electric operation, and the bridges so constructed as to care for all vehicular traffic, including probably auto-busses.

The possibility of traffic tunnels proving moderately successful does not alter the facts in the case of tunnel *versus* bridge, as there are enough physical, physiological, and psychological reasons against tunnels for automobile traffic to make it unnecessary to calculate relative costs. Tunnels are best suited for handling electric rapid transit and bridges, for automobile traffic. Thus, it is easy to see the logical solution of the problem in New York—to construct additional tunnel connections for the subways across the Hudson and East Rivers, so as to remove all tracks from the East River bridges, which will then have enough additional carrying capacity to care for automobile traffic for some years to come; then to construct two bridges over the Hudson River for automobile traffic alone, and three double tube tunnels for rapid transit. That a tri-borough automobile bridge over the East River will be needed eventually must be admitted. If the rapid transit tracks are removed from the old Brooklyn Bridge, it can be remodeled at a cost of less than $15 000 000 to care for a large portion of the down-town automobile traffic.

The construction of facilities on this general scheme will solve the problem of river crossings in a most economical and satisfactory manner for other cities. Most of the large cities in this country are suffering from a plethora of ideas and of city commissions. The creation of a General Utilities Commission, with departments to handle each particular civic problem, would make possible a real co-ordination of all the public utilities of a city, and also make it possible to carry out the necessary construction in a logical and progressive manner, so as to keep pace with the public need for the most urgent improvements.

M. W. Weir,* M. Am. Soc. C. E.—Most people will do what is easy, convenient, and time-saving, both from the standpoint of physical and financial effort.

A bridge becomes an important link in the transit systems of the places it connects. Its importance lies in the fact that it has provided the means of co-ordinating the transit systems and traffic lanes on both sides of the physical barrier which it overcomes, and its full effect on the population of the satellite city is not manifest until this co-ordination is so far completed as to make it both physically and financially easy, convenient, and time-saving for the public to use the bridge.

This paper, in noting the discrepancy between the increase in population of Brooklyn and the increase in passenger traffic across the river, registers surprise; but this relation is accounted for by the fact that practically all the people that took up residence in Brooklyn, became potential passengers—in fact in going to New York and back each individual became in the records two passengers—and to these must be added the traffic due to business created by the increased population on both sides of the river.

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It is interesting to note in Fig. 2* that the bridge traffic has a general recession from 1907 when the Interborough Tunnel was opened, which may show the influence of still another element that only began to demonstrate itself materially at about this time—the automobile. The frantic struggles of trolley companies to keep alive during the past few years, even with increased fares and more or less neglected maintenance, cannot be laid to increased population, but must be accounted for by the increasing use by the public of the automobile and the bus. In all these cases the new method has succeeded because it is easy, convenient, time-saving, and financially economical.

Conclusions drawn from past performances where all the conditions bearing on the question are fairly constant, may be relied on to a large extent in forecasting a future performance, but to draw conclusion from the past performance of the transit over Brooklyn Bridge, or any of the other East River bridges, either singly or collectively, is apt to be misleading, unless such a forecast pertained to a bridge surrounded by similar conditions; and while in another place the physical conditions may be the same, the habits and temperament of the people and their occupation may be so different as effectually to upset a calculation based on one comparison.

In comparing the traffic to be expected and the method of handling it over the Philadelphia-Camden Bridge, the physical conditions on the Pennsylvania side of the river are immeasurably different from those on Manhattan Island. There is no barrier like the Hudson River to shut off the expansion of Philadelphia to the west, no barrier like New York Bay to shut off expansion in a southerly direction. In fact, Philadelphia is free to expand in every direction except eastward.

For unexplained reasons cities show a decided tendency to expand toward the west when, as at Philadelphia, no barrier exists to thwart this trend. With ample room to grow, it only remains for rapid transit of one type or another to stimulate that growth, for it is manifest that city people no longer calculate distance in terms of miles but in terms of minutes.

This physical difference between Philadelphia and Manhattan is sufficient in itself to upset a forecast based on past performances of Manhattan-Brooklyn Bridges. In addition there is the further difference that Manhattan is a great harbor port and selling center where comparatively no large manufacturing plants are operating, while Philadelphia is a manufacturing city located many miles from the sea.

Brooklyn has been termed the "city of homes, rubber plants, and baby carriages," while Camden is an industrial center; furthermore, Brooklyn is a seaport of importance, while Camden is a river port many miles from the sea. The occupations of the people correspond to the difference between a manufacturing center and a selling center.

The effect of the Philadelphia-Camden Bridge on the vehicular traffic over the Market Street and Vine Street ferries will probably be noticeable in so far as light vehicles are concerned, particularly on Sundays and holidays when the traffic to Atlantic City is heavy. Truck traffic originating along the river

* *Proceedings*, Am. Soc. C. E., April, 1924, p. 392.

front will probably continue to use the ferries to a considerable extent for economy's sake.

Passengers to trains in Camden, except those originating within a short distance of the Philadelphia end of the bridge, will find but little advantage in the bridge. Passenger traffic in either direction over the bridge would become of moment only if continuous transportation to the environs of the two cities is accomplished with an expedition, convenience and economy, at least equal to that of the channels now in operation.

Of the nearly 4 000 passengers arriving in Camden from Philadelphia in the morning (Fig. 5)*, 3 000 were pedestrians, largely workers in the adjacent Camden factories; the same chart shows that in the evening, of the 7 000 passengers, less than 1 000 walked from the ferry. This indicates that more than 6 000 lived at a distance from the ferry; personal observation would lead to the belief that the number traveling to the suburban villages just outside of Camden was large.

Shuttle service across the bridge would be of doubtful advantage to the public and of precarious existence financially as it would have such a limited area for supplying passengers. This is believed to be true of all the bridges.

It is doubtful if either the trolley companies of New Jersey or those of Pennsylvania will be at all interested in crossing the Delaware River Bridge. Trolley companies all over the eastern part of the country must feel that their existence is seriously threatened. The recent strike of trolley employees in New Jersey demonstrated that motor bus transportation, properly developed, would have decided advantages; for instance, the busses do not occupy fixed positions in the street, they can pick up and deliver passengers at the curb, and they do not present an obstacle to street traffic as does the trolley car. The capacity of the street is vastly increased by eliminating the trolley car—a matter of considerable importance to cities with narrow streets—and the busses are rapid and not subject to delays originating in preceding vehicles. Furthermore, the attitude of the public is also a strong argument against the trolleys. Taking all these things in consideration, it would be a doubtful venture for trolleys to undertake a program of extensive betterments and extensions, particularly so when it is considered that crossing a bridge like the Delaware River Bridge, entails construction and maintenance for a considerable distance without creating any additional points for passenger collection. As a business proposition it is not enticing.

Operation over the bridge of a transit line which reaches not only into Camden but well beyond into the suburbs might induce Philadelphians to live in New Jersey provided they could do so conveniently and more cheaply than in Pennsylvania, particularly if, after reaching Philadelphia, they could continue to their destination without change. Such an arrangement entails of course all the difficulties of interlocking corporations, political boundaries, and human elements, pugnacious and irritating. That it is not impossible to attain such perfection of transit is exemplified by the subway systems operating in Manhattan and Brooklyn.

* *Proceedings*, Am. Soc. C. E., April, 1924, p. 398.

At this time surface transit systems are so threatened by more modern methods that it is doubtful if former accomplishments can be used as a basis for speculation.

The Delaware River Bridge is best fitted to serve its primary purpose, namely, to accommodate vehicular transportation, of which the Sunday and holiday traffic from Philadelphia to New York and from Philadelphia to Atlantic City will be the outstanding feature. It will be Pennsylvania's gateway to the sea.

HERMAN H. SMITH,* M. AM. SOC. C. E.—The study of the tremendous increase in the traffic between Brooklyn and Manhattan is interesting and involves consideration of certain changes in the economic life of the two communities which, aside from the growth in population, has had a direct effect on this volume of interborough traffic.

The author has pointed out that during certain periods the traffic across the East River has increased, relatively, much faster than the population of Brooklyn; and from this fact he draws the conclusion that the reduced cost and increased convenience of transportation over the bridges created a large volume of new traffic. While unquestionably the best and cheapest service will attract the most traffic, the conditions in New York are such that great care must be exercised in drawing conclusions of a general character from the results here obtained, if it is intended to apply such conclusions elsewhere.

The Island of Manhattan is the great business, commercial and industrial center of the City of New York and its environs. The island's commercial growth has curtailed its residential growth and many former residents of Manhattan now help to swell the tide of daily traffic which surges back and forth between it and Brooklyn.

A tremendous business and industrial expansion is taking place continuously on Manhattan Island; it is quite evident that this growth requires each year more and more workers, whom Manhattan cannot supply, and who must, therefore, come from the surrounding territory. These conditions have caused an abnormal increase in the population of the adjacent communities, as compared with Manhattan itself. It is this great outside population, a large percentage of which is dependent on the business and industry of the metropolis which supplies the bulk of workers for Manhattan's commercial activities. Of all the other Boroughs, Brooklyn lies nearest, with a population about equal to that of Manhattan, so that here exists a condition probably unequalled as an inter-community traffic problem.

Another economical condition has also entered into the traffic problem and has helped to increase this interborough traffic at a much greater rate than the increase in population alone would indicate. At the time the Brooklyn Bridge was opened, about forty years ago, comparatively few women who lived in Brooklyn were engaged in business or industry in Manhattan. Subsequently, each year has seen more and more women thus employed, so that now a much larger percentage of the population of Brooklyn is traveling back and forth each day between the two boroughs. Aside, therefore, from the fact that better

* Deputy Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

and cheaper transportation has tended to create more traffic, this change in the economic condition has probably been the main factor in increasing the traffic at a faster rate than the population.

Whether or not the lessons which New York has taught with respect to inter-community traffic can be applied to good advantage in connection with the construction of bridges in other communities, will depend on local conditions, and the judgment used in applying the results. Manhattan is an island surrounded by a population three or four times as great as its own population. The population of Manhattan cannot grow to any appreciable extent, whereas its business and industry are expanding at a rate which makes it necessary to draw more and more on the surrounding territory for its workers. This condition is probably unparalleled in any other community; consequently, it is easy to be misled by the traffic growth which has resulted from this unique situation.

D. B. STEINMAN,* M. AM. SOC. C. E.—The analysis of the traffic development on the Brooklyn and Williamsburg Bridges, presented by the author, is instructive and interesting, and his conclusions appear to be logical deductions from the data submitted. His principal conclusion, to the effect that rail facilities on a bridge are best utilized as the extension of the transit lines in the minor of the two communities connected by the structure, will hardly be disputed.

It is to be regretted that the author has focussed attention chiefly on what is properly the secondary division of traffic on highway bridges, namely, the rail traffic. It would be extremely desirable if he could present similar traffic curves and conclusions in regard to the major division of highway bridge traffic, the vehicular or automobile traffic.

In connection with the Brooklyn Bridge, his traffic curves show a development of the volume of passenger traffic to the amount of 300% of the maximum passenger traffic on the ferries prior to the construction of the bridge. In the case of the Williamsburg Bridge, the volume reached was 270% of the previous maximum ferry traffic. It would be desirable if, in his closing discussion, Mr. Miller could give similar relations for the vehicular traffic on the bridges as compared with the previous vehicular traffic on the ferries.

The author's arguments have been principally directed to the question of the best use that can be made, after the completion of the bridge, of rail facilities provided on the structure. A somewhat different angle of the same problem is the question of the conditions under which it is justifiable or advisable to provide rail traffic facilities on a highway bridge.

In connection with recent studies for various bridge projects, the speaker has come to certain conclusions on this question, and the facts and figures presented in this paper seem to bear out those conclusions. In his discussion of the best use of the tracks on the Delaware River Bridge, the author has shown that their utilization for shuttle service would be neither popular nor profitable; he has shown that the use of the tracks as an extension of transit lines now existing in Philadelphia would result in a loss of revenue to the operating companies; and he has suggested, as the most promising solution of the prob-

* Cons. Engr., New York, N. Y.

lem, the utilization of those tracks for extensions of present transit lines on the Camden side. Even with the last solution, however, he does not promise any positive profit to the operating company.

This questionable showing as to profit for the operation of the tracks on the bridge emphasizes the importance of the investment represented by these rail traffic facilities. Of the total live load calculated for the Delaware River Bridge, 1 000 lb. per lin. ft. represents foot traffic, 4 000 lb. per lin. ft. represents wheel or vehicular traffic, and 7 000 lb. per lin. ft. represents rail traffic. Of a total of 12 000 lb., 7 000 lb. represents the provision for the traffic on the four tracks. In other words, about 60% of the live load capacity of the bridge is provided to take care of its anticipated rail traffic. It would appear, therefore, that if the tracks had been omitted from the structure the total cost might have been reduced 40 or perhaps as much as 50 per cent.

This investment in the initial cost of the structure should be considered in arriving at any judgment as to the economic justification of providing these tracks on the Delaware River Bridge. It is a question whether any company would undertake to operate those tracks if, in addition to its ordinary operating expenses, it would have to pay interest on the investment represented by the track provision in the first cost of the structure.

Similar conclusions can be drawn from studies of other bridges. There are a number of toll bridges crossing the Ohio River, which provide for both wheel and rail traffic. The speaker recently had occasion to go over the figures of earnings and expenses on those bridges, and was surprised to find that of the gross earnings only 20 to 30% was produced by the rail traffic; and this, in spite of the fact that the rail traffic represented the larger proportion of the first cost of the structures, also the greater proportion of the annual charges for maintenance, repairs, and depreciation.

These facts certainly place a question mark against the economic advisability of providing for rail traffic on highway bridges. Highways are used primarily for vehicular traffic, and a highway bridge is merely an extension of the highways, a connecting link to close an existing gap; and it would appear logical, therefore, to use the bridges for the same kind of traffic that dominates the highways of which the bridge is a continuation.

The problem may be somewhat different in the case of a large city bridge, but even in that case similar conclusions are justified. In a city bridge, it may be desired to provide for continuation of the trolley or rapid transit service found in the city streets. Trolley service, however, is rapidly being superseded by the more modern system of bus service, which appears to be the best solution under conditions of growing street congestion. Rapid transit service in cities is either in subways or on elevated tracks. Elevated construction is coming to be regarded as obsolete; the City of New York now contemplates going to a great expenditure of money to eliminate the existing elevated structures as obstructions to light and traffic. For the rapid transit lines carried in subways, it would appear more logical to carry the traffic across the river in tubes. Accordingly, in cities, as well as in rural districts, there appears to be little economic justification for placing rails on a highway bridge.

There is another phase of this question: In promoting a bridge project, the most difficult problem is not that of engineering, but of financing, and any overloading of the financial end of the problem makes it more difficult to realize the project; that fact should be kept in mind before burdening the project with an element of questionable economic justification.

In conclusion, the speaker submits that a highway bridge should be built primarily for the natural traffic of the highways, which is vehicular traffic, and that the increased investment required for rail traffic provisions will generally fail to contribute a commensurate increase in usefulness or revenue-producing value to the structure.

H. M. LEWIS,* M. Am. Soc. C. E.—The paper refers only to bridges of large span connecting communities each of which have their own business and residential centers. Limiting his discussion to this type of bridge, the speaker believes that, in general, trolley lines should be extended across the bridge from the smaller community to a terminal in the larger one. Heavy trains, such as are carried on elevated railroads or in a subway system, should be extended at least into the business centers of both communities. The minimum facility desirable at either end of the bridge would be a loop connecting, if possible, with another parallel bridge, distributing and collecting the passengers to and from several points. A terminal operation of bridge trains in a large city is apt to lead to such congestion as occurred at the New York end of the Brooklyn Bridge, and which has not been entirely relieved even by the addition of several other routes leading from Brooklyn to Southern Manhattan.

Bridges of large roadway capacity are apt to create serious street-traffic congestion at their terminals, particularly as the street system, generally laid out long before the bridge was planned, is almost certain to be ill-adapted for such approaches. The farther rail passengers can be discharged from this point of street congestion, the better it will be for general conditions.

The effect of bridges on transit conditions depends not only on the location of the bridge in reference to existing cross-river facilities, but may be vitally influenced by the following factors:

1.—The topography of the shores and their adaptability to the economical construction of transit connections.

2.—The type of transit system, existing or proposed, in the communities which the bridge connects, that is, whether it is a surface, elevated, or subway system.

If there is low land on both shores, the provision of approaches for rapid transit lines becomes an expensive item, but trolley connections and terminals can be installed much more easily. Where the shores are relatively high, connections with existing rapid transit lines, either elevated or subway, is greatly simplified. The ease with which such connections can be made thus becomes important in determining the feasibility of carrying transit facilities across a bridge.

In New York, the East River bridges will undoubtedly have less and less effect on transit conditions, as the subway tunnels will become more and more

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the main passenger carriers. If it is ever possible to abolish the elevated railroads in down-town Brooklyn (surely desirable and quite possible in the next generation), the Brooklyn Bridge will become still more negligible as a transit factor. The Manhattan Bridge, due to its strategic location and great capacity, has largely superseded the Brooklyn Bridge as a transit carrier, while the Williamsburg Bridge, directly connecting the most densely populated parts of Brooklyn and Manhattan, carries far more transit passengers than any of the other East River bridges. Counts made by the Department of Plant and Structures, in November, 1922, indicated that the total passengers carried annually by the trolleys and the elevated railroad and subway cars on the East River bridges were distributed about as follows:

Williamsburg Bridge	149 000 000
Manhattan Bridge	96 000 000
Brooklyn Bridge	49 000 000
Queensboro Bridge	25 000 000
Total	319 000 000

It surely is hardly fair to classify bridges that carry such a large number of passengers on transit lines primarily as highway bridges.

There is a rapid transit tunnel now almost directly beneath the Queensboro Bridge and there will undoubtedly be future tunnels very close to the other bridges.

Because of the high shores on the New Jersey side, the Hudson River Bridge, if built, would be especially adaptable for transit facilities. If this bridge were built at a point in Upper Manhattan, such as Fort Washington Park, where the New York shores are also high, connections on both sides would be relatively simple. Transit facilities at this point, however, should form part of a circumferential transit line, as it is too far north to serve as one of the principal arterial connections to Lower Manhattan, and the use of it in this connection would throw too great a burden on existing or proposed north and south routes within the city. If facilities could be provided for by-passing the congested parts of Manhattan and connecting Northern New Jersey with the apartment section of Manhattan and the Boroughs of the Bronx and Queens, the increased facilities for connection between these outlying parts of the city would not only save long delays for persons now wishing to pass between those points, but would facilitate a decentralization of business activities.

In response to Mr. Fowler's question as to the total figures for subway traffic between Manhattan and Brooklyn, it may be of interest to note that the Engineering Division of the Regional Plan of New York and Its Environs recently compiled some figures showing the distribution of passenger traffic between Manhattan—south of 59th Street—and the surrounding areas. These indicated that the total number of persons that would probably enter (one-way traffic) this district during 24 hours on a typical business day in 1924 would be distributed about as follows:

Brooklyn-Manhattan Transit and Interborough Rapid Transit Tunnels to Brooklyn	318 000	
Brooklyn-Manhattan Transit and Interborough Rapid Transit Tunnels to Queens	102 500	
Pennsylvania and Long Island Railroad Tunnels to Queens	48 500	
Total		469 000
By the four East River Bridges, including passengers by rapid transit, trolleys and vehicles, and pedestrians		628 000
By ferries on the East River, south of 59th Street		19 900

Assuming that a typical business day is one-three hundred and fortieth of the annual total, the corresponding 1924 totals for traffic both in and out would be 319 000 000 by tunnels; 427 000 000 by bridges; and 13 600 000 by ferries. Thus, it is seen that of the total passenger traffic between Long Island and Manhattan—south of 59th Street—56% is carried by bridges, 42% by tunnels, and only 2% by ferries.

T. Kennard Thomson,* M. Am. Soc. C. E.—The author is to be congratulated on his excellent paper. The increases in passenger traffic from Brooklyn since 1871 (Fig. 2)† is a reminder of the interesting paper‡ by F. W. Gardiner, M. Am. Soc. C. E., in which it was stated that the Elevated Railroad Company had advertised, in 1872, that it was carrying 1 300 passengers per day and had solved the transit problem for New York. This compares with the 2 500 000 passengers now carried daily on all transit facilities combined.

About twenty-six years ago, the speaker published an article in *The Brooklyn Eagle,* in which he stated that the only way to solve the terminal problem in New York was to abolish the terminal.§

To build a bridge solely for highway traffic, as suggested by Mr. Steinman, either in New York or in Philadelphia, would surely be to serve the automobile owning class at the expense of the masses.

One question which has not been considered in the discussion, but which has been fully covered in the *Transactions* of the Society, is the damage to property along the water-front, due to the construction of the East River bridges. It will be interesting to note whether a similar result occurs in Philadelphia and Camden.

* Cons. Engr., New York, N. Y.

† *Proceedings,* Am. Soc. C. E., April, 1924, p. 392.

‡ "Manhattan Elevated Railway Improvements", *Transactions,* Am. Soc. C. E., Vol. LXXXII (1918), p. 553.

§ *Transactions,* Am. Soc. C. E., Vol. LXXV (1912), pp. 268–269.

ANALYTICAL SOLUTION OF MASONRY DOMES

Discussion*

By Messrs. Charles S. Whitney and William Cain.

Charles S. Whitney,† Assoc. M. Am. Soc. C. E. (by letter).‡—The ordinary theory of domes as expressed in Mr. Coyle's paper is based on the assumption that the dome is a free body cut away from its supports, which are replaced by vertical forces and ring tension. The lantern ring, if one exists, is also considered to be free, as though both lantern and support rings bear against the dome shell without friction. Under this assumption, it is possible for a high tension or compression to exist in the ring with a low circumferential stress even of opposite sign in the adjacent shell. This, of course, is not the condition in practice, where the rings are monolithic with the dome and the deformation of the rings has an important effect on the circumferential stresses, particularly near the rings. These stresses should be considered carefully although an exact analysis of them does not seem possible.

A stretching of the supporting ring or a yielding of the abutments has the effect of increasing the circumferential tension or decreasing the circumferential compression in the shell near the supporting ring and also of increasing the circumferential compression at the top. This means that the radial component of the circumferential stress is no longer great enough to confine the meridional thrust to the center of the shell and that this thrust will rise above the center near the support and produce positive bending, which might be very dangerous in a thin shell because it has a tendency to cause bursting. The tendency is to split the lower part of the dome into radial strips which act as buttresses. Clearly these buttresses will be most effective if they are straight. It is therefore advisable to increase the thickness of the shell near the supporting ring, making it conical in shape for a short distance. The lower part of the shell should be reinforced with sufficient circumferential steel to absorb the tension induced by the ring deformation.

If the dome is exposed to the weather, temperature changes may have an important effect, as the upper part of the dome is more exposed and may pass through a much greater range of temperature than the supports. A decrease in the temperature of the portion of the dome above the supports or a shrinkage of the concrete of the shell will have the same effect as a stretching of the supporting ring. This will tend to produce compression in the supporting ring with tension in the dome shell just above.

* This discussion (of the paper by David C. Coyle, Esq., published in *Proceedings* for April, 1924, but not presented at any meeting of the Society), is published in *Proceedings* in order that the views expressed may be brought to all members for further discussion.

† Cons. Engr., Milwaukee, Wis.

‡ Received by the Secretary, April 7, 1924.

A similar but opposite action is produced by the compression ring at the lantern unless the stress in the ring is kept as low as the circumferential stress in the adjacent shell.

In the case of the full hemispherical dome, according to the theory as stated by the author, the circumferential stress is greatest at the base while the stress in the supporting ring is zero. This of course would require that the dome be supported on a frictionless base. Actually, the circumferential deformation at the base of the shell must produce a swelling which will induce a tension in the supporting ring or compression in radial buttresses (if they are provided). The dome does not act as a full hemisphere but, as the paper states, as though the lower part were a conical surface.

For spherical domes with shells of uniform thickness Fig. 8 gives a convenient method of computing the stresses according to the ordinary theory and shows clearly the variation in their magnitude. The live load is assumed to be a full uniform load of p lb. per sq. ft. of horizontal projection.

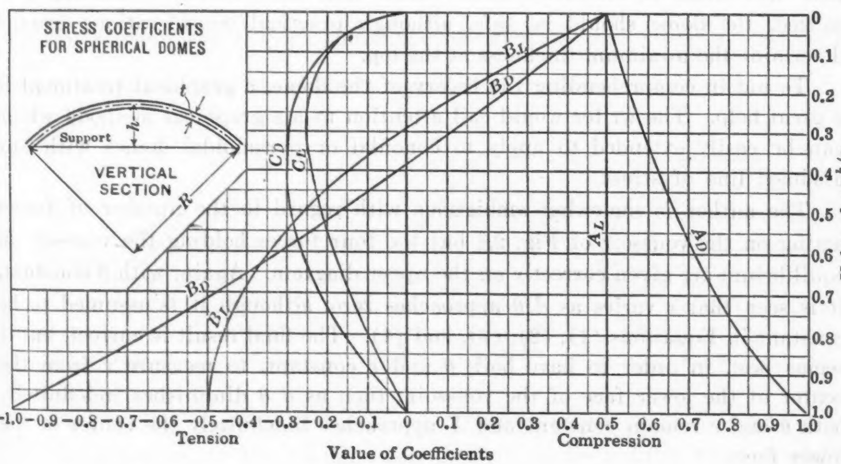

FIG. 8.

The stresses are given by the following formulas:

Dead load stresses:

$$\text{Meridional} \ldots\ldots\ldots A_1 = A_D\, R\, g$$

$$\text{Circumferential} \ldots\ldots B_1 = B_D\, R\, g$$

$$\text{Supporting ring} \ldots\ldots C_1 = C_D\, R^2\, g\, t$$

Live load stresses:

$$\text{Meridional} \ldots\ldots\ldots A_2 = \frac{A_L\, R\, p}{t}$$

$$\text{Circumferential} \ldots\ldots B_2 = \frac{B_L\, R\, p}{t}$$

$$\text{Supporting ring} \ldots\ldots C_2 = C_L\, R^2\, p$$

in which,

t = the thickness of the shell,

g = the heaviness of the material per unit volume, and

c = the ratio of the height of the dome to the radius of the sphere.

The values of A_D, A_L, etc., may be determined from Fig. 8.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).†—The author is to be congratulated on his very general and simple solutions of conoidal and pyramoidal domes for the assumed line of stress. The dome is, of course, an indeterminate structure, for which a strict solution can only be effected by consideration of the elastic yielding of its parts; but for domes of the usual thicknesses, it is thought that a practical solution can be made by assuming the line of stress to be on or near the center line of a meridian section.

The writer has already discussed rather fully such possibilities,‡ and he agrees with the author, that, using the formulas of this paper and an allowable unit stress in compression, not more than one-half that ordinarily permitted, the dome should be safe, although practical considerations usually determine the minimum thickness at the top.

To aid in comprehending the theory of the dome, a graphical treatment is a great help. The writer would call attention to his graphical method§ which can be easily extended to apply to conoidal or pyramoidal domes with any assumed line of stress.

The author is somewhat ambiguous with regard to the number of forces acting on the voussoir of Fig. 2,‖ but the four forces holding the voussoir in equilibrium are given correctly on the upper diagram. Again, with θ constant, it is seen that r varies as $d\,\theta$ approaches zero, although it is assumed to be constant in Equations (1), (2), (3), and (4). The final result is correct, but it seems best, in order to have both θ and r constant, to measure r from the center of the lower face of the voussoir; then as $d\,\theta$ diminishes indefinitely, both θ and r remain constant and X approaches indefinitely the center of the lower face.

A slightly different derivation of Equation (4), but along the lines sketched by the author, may not be amiss, as it checks his result. In Fig. 9(a), the voussoir (shown shaded) is represented as held in equilibrium by its weight, the two meridional thrusts, and the bursting pressure. In Fig. 9(b), the force diagram is shown. In this, the vertical, $O\,D = \frac{W}{2\,\pi\,r}$, represents the entire weight of dome, lantern, snow, etc., sustained by the upper conical face of the voussoir, and $DE = p\,R\,d\,\alpha$ represents the weight of the voussoir. The rays OA, OK, are drawn parallel to the tangents of the center line of the meridian section at the centers of the upper and lower faces of the voussoir and are limited at

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† Received by the Secretary, May 5, 1924.

‡ "Theory of Steel-Concrete Arches and of Vaulted Structures", pp. 156–215.

§ *Transactions*, Am. Soc. C. E., Vol. LV (1905), pp. 201–227.

‖ *Proceedings*, Am. Soc. C. E., April, 1924, p. 403.

A and K by horizontal lines drawn through D and E. A vertical, $KI = DE$, cuts off the "bursting pressure," $AI = H\,R\,d\,\alpha$, on the horizontal, AD.

The voussoir is thus in equilibrium under the four forces, OA, AI, IK, and KO, the magnitudes and directions of which are given by the closed polygon, $O\,A\,I\,K\,O$ (Fig. 9 (b)). For equilibrium, the algebraic sum of the horizontal components of these forces must equal zero. Therefore, $AI = AD - KE$, or,

$$H\,R\,d\,\alpha = \frac{W}{2\,\pi\,r}\cot\alpha - \left[\frac{W}{2\,\pi\,r} + p\,R\,d\,\alpha\right]\cot(\alpha + d\,\alpha),$$

$$H = \frac{W}{2\,\pi\,r\,R} - \frac{[\cot\alpha - \cot(\alpha + d\,\alpha)]}{d\,\alpha} - p\cot(\alpha + d\,\alpha).$$

H, in this case, is the average bursting force per unit of area acting on the voussoir. As $d\,\alpha$ approaches zero indefinitely, H approaches the intensity of the bursting pressure at X, the center of the upper face of the voussoir; and, as proved in any calculus, $\dfrac{\cot(\alpha + d\,\alpha) - \cot\alpha}{d\,\alpha}$ approaches indefinitely ($-\operatorname{cosec}^2\alpha$) as its limit; whence, taking limits and multiplying by r, to find the ring stress, $B = H\,r$, B, the ring stress at X, $= \dfrac{W}{2\,\pi\,r}\operatorname{cosec}^2\alpha - p\,r\cot\alpha$, which is equivalent to the author's Equation (4)*, since $\alpha = 90° - \theta$.

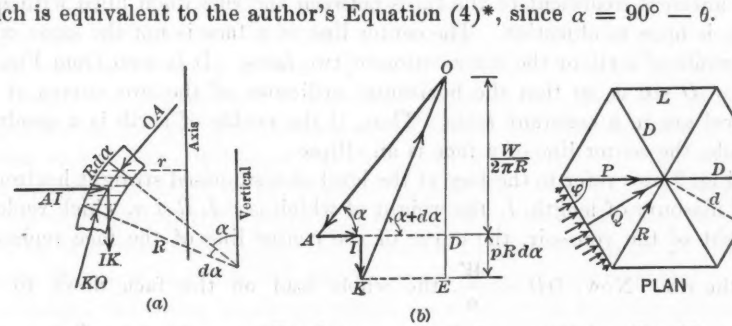

FIG. 9. FIG. 10.

The reference to the calculus can be avoided by a little trigonometrical reduction. Thus,

$$\frac{\cot\alpha - \cot(\alpha + d\,\alpha)}{d\,\alpha} = \frac{1}{d\,\alpha}\left[\frac{\cos\alpha}{\sin\alpha} - \frac{\cos(\alpha + d\,\alpha)}{\sin(\alpha + d\,\alpha)}\right]$$

$$= \frac{1}{d\,\alpha}\,\frac{\sin[(\alpha + d\,\alpha) - \alpha]}{\sin\alpha\sin(\alpha + d\,\alpha)} = \frac{\sin d\,\alpha}{d\,\alpha}\;\frac{1}{\sin\alpha\sin(\alpha + d\,\alpha)}$$

the limit of which, as $d\,\alpha = 0$, is

$$1\,\frac{1}{\sin^2\alpha} = \operatorname{cosec}^2\alpha$$

The equivalents of the author's Equations (5)* and (6)* are derived directly from Fig. 9 (b). Thus, the meridional thrust at X is,

$$O\,A = A = -\,\frac{W}{2\,\pi\,r}\operatorname{cosec}\alpha$$

* *Proceedings*, Am. Soc. C. E., April, 1924, p. 404.

and with the interpretation given to C,

$$C = A\,D \times r = \frac{W}{2\,\pi} \cot \alpha$$

In his paper on the "Theory of the Spherical or Conical Dome of Reinforced Concrete or Metal,"* the writer used diagrams like Fig. 9 in developing formulas for meridional and crown stresses and hoop tension, corresponding in meaning to the author's A, B, and C, and he was careful to point out that the graphical method, giving average values for B (or C), would differ slightly from values found analytically, which always refer to intensity at a point. His Formulas (3), (4), and (6), for the spherical dome† agree with the author's Equations (17), (16), and 18),‡ when the notation is made the same, a fact which the author seems to have overlooked when he states his belief that Equation (18) is original. In fact, Rankine's equivalent formula for "hoop tension" was published years ago.

In treating pyramoidal domes, Fig. 9 can again be utilized, letting $OD = \frac{W}{n}$ and $A\,I = \frac{BL}{D} R\,d\,\alpha$. Only a slight change in the preceding analysis is necessary to deduce the values of A, B, and C.

The author's treatment of the faces between the ribs when filled with solid masonry is open to objection. The center line of a face is not the same curve as the profile of a rib or the intersection of two faces. It is seen from Fig. 10, that $d = D \sin \phi$, so that the horizontal ordinates of the two curves at the same level are in a constant ratio. Thus, if the profile of a rib is a quadrant of a circle, the center line of a face is an ellipse.

Let Fig. 9 now refer to the face at the level of a supposed straight horizontal beam of masonry of length, L, the weight of which is $p\,L\,R\,d\,\alpha$, which replaces the weight of the voussoir, the curve of the center line of the face replacing that of the rib. Now, $OD = \frac{W}{n}$, the whole load on the face down to the level of the beam, and $DE = p\,L\,R\,d\,\alpha =$ weight of beam of height, $R\,d\,\alpha$, along the arc; then, in Fig. 9 (*b*), $AI = L\,H\,R\,d\,\alpha$, is the horizontal force acting at right angles to the beam at its center, as induced by the meridional thrusts. In this instance, H is the corresponding unit force, acting on a unit of length of beam and a unit of profile of the face.

From $A\,I = A\,D - K\,E$,

$$L\,H\,R\,d\,\alpha = \frac{W}{n} \cot \alpha - \left[\frac{W}{n} + p\,L\,R\,d\,\alpha\right] \cot (\alpha + d\,\alpha)$$

from which, proceeding as before,

$$HL = \frac{W}{n\,R} \operatorname{cosec}^2 \alpha - p\,L \cot \alpha$$

In this formula, HL represents the total horizontal force acting on the horizontal beam of length, L, and of one unit height along the profile of the face.

* *Transactions*, Am. Soc. C. E., Vol. LV (1905), p. 201.

† *Loc. cit.*, pp. 213–217.

‡ *Proceedings*, Am. Soc. C. E., April, 1924, pp. 407–408.

The author chooses to designate this force by the awkward expression, $\frac{BL}{D}$, for which Equation (8)* is identical with the writer's formula, except that the angle, α, of this formula refers to the profile of the face, whereas θ in the author's Equation (8) refers to the profile of the rib.

There are serious objections to the analysis for evaluating HL, for it tacitly assumes that the face is divided up, not by meridian planes but by vertical planes, all perpendicular to the horizontal beam, say, one unit apart. All the corresponding shells, of unit width, will have parallel profiles and parallel tangents, and their weights, for a full dome, will vary nearly linearly from a maximum at the center of the beam to zero at the ribs, and the unit horizontal forces acting on the beams will vary correspondingly according to the theory mentioned. There is now, however, shear along the divisional planes, so that the theory does not exactly apply. The effect of this shear will be to make the unit horizontal forces acting on the beam more nearly uniform.

This is seen, also, by dividing the shell of the face by meridional planes, between which there is, presumably, no shear, as there certainly is none for a conoidal dome. If a face of a pyramoidal dome is divided by meridional planes, all passing through its vertical axis, a voussoir of the horizontal beam contained between two such planes, will be in equilibrium under its weight, the two meridional stresses, and a horizontal radial force. This last force is not perpendicular to the beam except at its center, and its component perpendicular to the beam is less at the ends than at the center. The sum of all such components acting on the beam is equal to the force, Q. The components are not uniformly distributed, but the component at the end of the beam is not zero; so that the supposition that Q varies from a maximum at the center linearly to zero at the ends will give an excess moment at the center.

Although the writer would not advise the adoption in practice of the imperfect solution just given, still, it suffices to bring out the fact, not hitherto noted, that the unit forces acting on the horizontal beam are not uniform; and to give an excess moment at the center of the beam, they can be regarded as decreasing uniformly from a maximum at its center to zero at its ends.

If Q represents the sum of the horizontal forces acting on the beam corresponding to the latter hypothesis, then, if the beam is regarded as fixed at the ends, the moment at the ends is easily shown to be $-\frac{5}{4}\left(\frac{1}{12}\ QL\right)$, and, at the center, $+\frac{3}{4}\left(\frac{1}{12}\ QL\right)$. For a simple beam, supported at the ends, the moment at the center is $\frac{1}{6}\ Q\ L$. The degree of fixity at the ends must determine the choice of coefficients, which, for usual designs, may approximate $\left(\frac{1}{12}\ QL\right)$ at the center and ends, as assumed by the author.

* *Proceedings*, Am. Soc. C. E., April, 1924, p. 406.

For a pyramoidal dome of masonry, P (Fig. 10), may be regarded as the resultant of the forces, $\frac{Q}{2}, \frac{Q}{2}$, acting on the lengths, $\frac{L}{2}$, of the horizontal beams on adjacent sides of the perimeter.* Thus,

$$2\left(\frac{1}{2}\ Q \operatorname{cosec} \phi\right) = P \therefore Q = P \sin \phi = P\frac{d}{D}$$

or, since, in the author's notation, $p = \frac{B\,L}{D}$, by the use of his Equation (8),

$$Q = \frac{d}{D}\left[\frac{W}{nR} \sec^2 \theta - p\ L \tan \theta\right]$$

To the fiber stresses at the ends and center of the horizontal beam due to the moment of, say, $\pm\frac{1}{12}\ QL$, must be added that due to the longitudinal stress, B.

The author has deduced the formulas for hoop tension, ring, and meridional stresses, for the conical dome from the corresponding formulas for the spherical dome, by making R equal to infinity.

All these formulas, like the corresponding ones for the conoidal and pyramoidal dome, are correct and in practical form. It is a pleasure to commend the paper as a good, brief, practical treatise on the interesting subject of the dome.

* As given by Mr. E. Schmitt in his paper entitled "Theory of the Spherical Dome with a Homogeneous Surface, and of the Framed Dome, also Notes on the Construction of Masonry and Metal Domes", *Transactions*, Am. Soc. C. E., Vol. LII (1904), p. 293.

THE ECONOMICS OF HYDRO-ELECTRIC DEVELOPMENT

Discussion*

By Messrs. L. F. Harza, Harry A. Hageman, C. P. Dunn, Charles B. Hawley, W. S. Lee, and Brent S. Drane.

L. F. Harza,† M. Am. Soc. C. E. (by letter).‡—The author has emphasized the economics of the isolated hydro-electric project designed to serve a given power market with such auxiliary steam service as is needed to insure continuity and reliability of service. The opportunities for the development of isolated projects to serve a given market are fast disappearing; interconnection of power systems, even of competing companies, is coming rapidly and inevitably.

In certain localities—such as the Niagara District, the area served by the Southern Power Company, the Pacific Coast District, the mining districts of Northern Michigan and others—hydraulic power as a source of energy predominates and steam power serves as auxiliary. As such systems become larger the problem of what output can be economically derived from a given hydro-electric project becomes more and more a question of how the flow of the stream will blend with that of others already developed and feeding into the system.

The writer has in mind a property in the Middle West where several hydro-electric projects serve a mining district. All the streams are in flood at the same time and the heads of the several projects are high enough so that the flood season is the period of greatest capacity. A new project was projected to feed into this system but little value could be given to secondary energy without a definite contract for the purchase of this energy, for the reason that this and all of the previously existing hydro-electric projects feeding into the system offered surplus power during the same season. The new project, therefore, had to be predicated largely on its low flow capacity.

There is a situation on the Pacific Coast which is of National importance as to the ultimate future. California is developing its hydro-electric power in rapid strides and is dependent largely on storage during the dry months in the summer and fall. In the Pacific Northwest States similar stream characteristics exist, although development is not taking place so rapidly. On the Columbia River in Oregon and Washington, there are large available power sites.

* This discussion (of the paper by Daniel W. Mead, M. Am. Soc. C. E., presented at the Spring Meeting, April 9, 1924, and published in *Proceedings* for April, 1924), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Hydro-Elec. and Hydr. Engr., Chicago, Ill.

‡ Received by the Secretary, April 8, 1924.

This river is remarkably well adapted for supplementing the existing projects of the Pacific Northwest and of California also, if the economy of long-distance transmission ultimately brings this market within its reach. Its sites offer secondary power during the summer and fall when the smaller streams of the region are very low.

The Columbia River is of such large capacity as to be able to supplement the smaller powers of a large region. If the prime capacity of the Columbia River site is taken as 500 000 h. p., then there would be an available surplus from about July 1 until January 1, varying from 0 to about 500 000 h. p. and back again. This period of surplus power of the Columbia closely fits the deficiency period of the other streams.

This is one of the best illustrations known to the writer of the ultimate possibilities of blending the capacity of streams of different characteristics in the same region to provide economical power far in excess of what any one stream could justify if developed independently.

In the Mississippi Valley region, where population is dense and industry highly developed, heads are low, often disappearing in time of flood, and the total of hydraulic power possibilities is small in proportion to the developed steam capacity of the region. In such districts a hydraulic development must often find justification purely from the standpoint of its saving in coal for that portion of the year when the power is available.

The tendency toward high capital cost of low-head developments, together with the seasonal characteristics of some of the low-head streams, might seem to preclude hydro-electric development in this region. The fact is, however, that because of the high industrial development from steam power, an unlimited market often exists for the hydro-electric energy which can be furnished when and if available. This justifies a development of the projects to a point much further in excess of the minimum flow capacity and thereby makes available a larger amount of energy from the project than would be justified in a region dependent largely on hydro-electric sources from rivers of similar type. The fact is that many of the small and low-head developments of the Middle West are thus fully justifying themselves and proving economically profitable.

The advent of the automatic hydro-electric station has served to warrant the economical use of many small power sites which would have been entirely unprofitable under manual operation. In some of the smallest of these developments the elimination of the cost of attendance has saved as much as 6% or 8% per annum on the capital cost, thus carrying the interest charge. The saving expressed in the percentage return on capital investment by virtue of automatic operation, varies almost inversely with the size of the station—it is of small importance to a large station, while it may be the saving which justifies the development of a small station.

The writer's experience in the development of several automatic stations and his observation of subsequent results, convinces him that the small automatic station will be one of the big fields of development of the future. To each big power site that attracts the attention of the utility company there are many small ones available but largely ignored to date.

HARRY A. HAGEMAN,* M. AM. SOC. C. E. (by letter).†—This paper furnishes a broad and concise outline of the many complex and often inter-related factors to be considered in determining the economics of a proposed hydro-electric development. Many of these factors have been discussed in detail in various engineering books and disconnected articles; it is a great boon to the engineer to have the problem completely outlined as in this paper.

The writer believes that it is worth while to emphasize a few of the topics about which comparatively little has been written.

Theoretically the complete development of a hydro-electric project should utilize all the energy at the site that can economically be controlled and marketed. The exact determination of the proper capacity is, however, extremely difficult and complex on account of the many factors involved.

A hydro-electric development is not different in principle from any commercial manufacturing business. The problems may be classified as (1) those pertaining to the supply of the raw material, or water supply; (2) those pertaining to the process of manufacturing, or the economical design of the plant; and (3) those pertaining to the disposal of the manufactured product, or the market. These general groups can be further sub-divided into the many factors mentioned by the author. All three are equally important, but either because of lack of experience or because precise engineering data may be difficult to obtain, inadequate consideration often is given to at least one of these general groups.

The initial capacity for a development is sometimes fixed by the immediate power demand with some allowance for growth, and sometimes largely by financial limitations. The total cost of the initial installation is often affected greatly by the layout requirements of the ultimate developments, and by the inclusion in the initial development of certain items of construction not required immediately. Because of the rapid changes in the art, only so much of the ultimate plant should be provided for as may be absolutely necessary to allow for future expansion at a minimum of expense. The details of the design and construction of later extensions should be left for future consideration.

If the proposed plant is to be one unit of a large power system, rather than isolated, the problem is further complicated. In the former case, the object is to increase the capacity and output of the system to a maximum amount at a minimum cost within the limitations of the market. In some cases the object may be to replace more expensive sources of energy or to insure continuity of service. In the case of the isolated plant, the object is to supply a pre-determined market, with proper allowances for growth during a reasonable number of years, at a price which would be attractive to prospective customers and still at a profit which would seem ample to the owners.

The success of the project is obviously dependent on the extent to which its output can be marketed. The demand for power usually varies with the hour, the day, and the season, and must be met immediately. As the supply of power is dependent on the supply of water, it is necessary either to have a

* Hydr. Engr., Stone & Webster, Inc., Boston, Mass.

† Received by the Secretary, April 9, 1924.

natural stream flow adequate for all demands, or to regulate this stream flow to the extent necessary that it may furnish an ample supply of power when the power is required. In other words, the important problem is to market the greatest possible proportion of the total available energy, and at the same time to insure a generating capacity equal at all times to the probable load requirements.

It is best to deal separately with the problems of demand and supply. Whenever practicable the simplest method is to determine first, the exact character of the load to be carried; and second, the means of securing the required power. When the load is nearly constant it is necessary either to secure a uniform stream flow or to equalize a variable flow by the use of storage. When the load varies, the stream flow must be either partly wasted, or controlled and regulated in accordance with the variations in load.

The term "load factor" conveniently expresses the ratio of the average load to the maximum for any given period of time such as a day, week, month, or year. It should be remembered, however, that this factor does not define the shape of the load curve, which becomes an important factor in a study of the amount of pondage required. A load that has a very sharp peak but is fairly constant for the remainder of the day will require but a fraction of the pondage necessary for one that has the same peak for several hours but is negligible at other times.

The character of the load determines the form of the load curve, which should be determined as accurately as possible, for both the present and near future. In general, its use will give more accurate results than the use of percentage load factors. On the other hand, some investigations do not warrant more than broad assumptions of probable load factors.

There is a tendency at most power plants for the load to repeat the same seasonal cycles so far as the shape of the load curves is concerned, except that an increase in the load will raise the curve. For this reason it is frequently convenient to plot load curves in terms of the percentage of the peak load rather than in terms of the actual power.

In meeting commercial requirements, it is the usual practice, after estimating the probable load curves or percentage load factors, to determine how these requirements can best be obtained with the stream flow available. The isolated plant presents quite different problems from those of a plant combined in a system of other water or steam plants; it is relatively simple, especially when the minimum stream flow can be definitely determined. There is then available a certain amount of dependable continuous power, known as primary power, which can be readily estimated after taking into account the amounts of pondage and storage available. In addition, during periods of ample stream flow, there may be excess power, variable in amount and largely uncontrollable, known as secondary power.

Where the proposed plant is to be part of a large power system, special studies are required to determine the most economical combinations. Generally speaking, a plant having large storage should be operated as a peak-load plant during periods of low water and as a base-load plant during periods of ample water. Where several plants in a system have considerable pondage and

storage available, it may be necessary for the new plant to operate on an intermediate portion of the load curve, being neither a base nor a peak-load plant, with the result that there may be some unavoidable waste of water which would not occur if it were an isolated plant. It is sometimes possible to secure diversity of stream flow, and to combine two hydro-electric plants having low flow periods at different times of the year, or having characteristics which permit the power from one to supplement that from the other.

A combination that is often particularly effective can be obtained by connecting a low-head plant located on a large river, with a high-head storage development on a tributary entering the main stream above. By this arrangement the bulk of the load during periods of high water is carried by the low-head plant, while water is being stored in the reservoir of the plant above. During low water the situation is reversed, a greater load being generated by the high-head plant using storage water, which also serves in a lesser degree to supplement the output of the plant below.

Whether the plant is to operate as a base or a peak-load plant materially affects both the total capacity and the size and number of units to be installed. If, in the future, the proposed plant is likely to be changed from one type to the other, the initial development should be designed and constructed so that the necessary plant alterations can be made without serious changes in existing structures.

In designing the initial plant, it may be that provision for future extensions would require expenditures that would make the first installation uneconomical, in which case it is proper to charge future additions with the increased expenditures rather than to make provision for them at the beginning.

The initial construction should always allow the greatest possible latitude for future extensions, as the details of the load and the design may so change between construction periods that units of different size and type may be more economical for the later installations.

If a power market must be created, or developed to use the output from a proposed water power, the venture is quite speculative, although careful engineering and financial studies will materially reduce the hazard. The same thing may be said as to the many physical difficulties to be reckoned with, such as the interference with the continuity of the power supply by trash, silt, logs, and ice; or the enforced release and perhaps waste of water for navigation or log driving; or the interference from log or ice jams, frazil and anchor ice; or even the winter freeze. Similarly, the contingency of labor strikes either during construction, or later in industries using the power, must be considered in order properly to minimize the hazards.

The past fifteen years have been years of recession in the cycles of rainfall and consequently of run-off. It has sometimes happened in a cycle of decreasing yield that the stream flow has been lower after the completion of a plant than short term records have predicted. This, of course, results in a lower output of power and energy and a longer time to fill ponds or reservoirs than was estimated, especially if water must be released on account of other plants down stream the supply of which cannot be entirely cut off.

In closing, attention is called to the importance of permitting the engineer to follow the operating conditions at the plant, both for the purpose of allowing him to check his power estimates or to improve on his design in future plants, and for the purpose of insuring that the plant is being operated efficiently—a condition that is not always attained and for which the engineer is unjustly criticized. It is sometimes advisable for the engineer to prepare operating curves and tables which will permit the operator of the plant to ascertain readily the best combinations of units and equipment for minimizing power losses. Certainly operating statistics are to be recommended, even if some expense is involved, in order to keep the proper control of such matters, as they can always be justified when plant extensions or improvements come up for consideration.

C. P. DUNN,* M. AM. SOC. C. E. (by letter).†—The writer has been impressed by the importance of the fundamental ideas set forth by Professor Mead, and believes that every engineer connected with hydro-electric design, even though his particular duties may appear to him to be purely technical, would do well to familiarize himself with the economic phases of the problem as outlined in this paper. Questions of cost and earning power are so interwoven with the scientific phases of design and construction that it is impractical to attempt to separate them in any study or discussion.

At present, men with a clear understanding of the technique of design and construction have an opportunity to broaden their activities by familiarizing themselves with the business problems encountered in building and operating electric utilities.

While the author's paper seems to follow, in general, the idea of studying an independent development, the principles enumerated are, of course, applicable to the much more common problem, the extension of an existing system.

The writer wishes to discuss certain points mentioned in the paper that deserve greater emphasis. The engineer often fails to make a dependable cost estimate, due to under-estimating the hazards of construction, to overlooking many small items which total up to large sums, to under-estimating overhead and general expenses, and, in short, to a lack of appreciation of the problem before him.

The engineer undertaking to estimate the cost of a plant amounting to millions of dollars is taking on himself a grave responsibility; the decision to build or not to build may be based on his recommendation, and the efforts of hundreds of men for several years may be exerted wisely or foolishly, depending on the clearness of his vision. He must put out of his mind the desire (which is ingrained in every engineer) to see the project with which he is connected under construction, and must weigh the problems before him with unbiased judgment.

It too often happens that the theoretical calculation of the kilowatt-hours that a proposed hydro-plant may generate, is allowed to intrude in an engineer's report without sufficient study to determine whether or not all these kilowatt-hours are usable within the limits of the daily and annual load curves

* Designing Engr., Portland Ry., Light & Power Co., Portland, Ore.

† Received by the Secretary, May 6, 1924.

of the system to which the plant is connected. A careful examination sometimes discloses the fact that not more than 60 or 70% of the kilowatt-hours originally thought available are actually usable or salable.

The forecasting of future daily and seasonal variations in the demand for power, and the study of characteristics of the forecasted curves in connection with the power available at the plant, are very important. The value of a kilowatt-hour lies not in the mere fact that it can be called a kilowatt-hour; its value is measured by whether or not it can be delivered on demand at any hour of the day or at any season of the year. A surplus hydro-kilowatt-hour at 2:00 A. M. in the wet season of the year may be compared to a cake of ice in midwinter—its value on the market is zero.

The foregoing emphasizes the difference in value between the power generated by plants which depend on stream flow as it comes, and those which are more fortunately situated in regard to daily pondage or seasonal storage.

When there is a choice between two or more sites for a hydro-electric development, the needs of the system to which the plant is to be connected for base power or for peak power, will have a bearing on the economic value of the various sites with respect to that particular system, and will influence the choice. In general, those sites which require the construction of long conduits under high heads are better suited to development as "base load" or continuous 24-hour per day plants, and those sites which permit the location of the power house very close to the dam are most suitable for "over-development" or "peak-load" plants, providing there is sufficient pondage available to cover the daily load fluctuations on such a plant.

It is very important that stream-flow measurements be made at a contemplated site over as long a period as possible. At times, power companies find themselves compelled to make developments on streams where gaugings have been carried on for only short periods and little is known of the characteristics of the stream. An erroneous estimate of the dependable stream flow may be very costly.

Other things being equal, several small steps in development, made as the demand for power requires, are better than one large step. This is because of the fixed charges on the large step during the long period in which it is only partly loaded. Fixed charges during the "development" or loading period may easily add 50% to the cost of a plant. The writer has pointed out a graphic method of presenting this phase of the problem.*

The solution of the economic size of a development as presented in the paper† is applicable only to a specific case. In the solution it appears to be assumed (1) that the demand for power will happen to be exactly the same as the amount of power that will be delivered by the most efficient combination of steam and hydro-electric plants; (2) that the load will not grow; and (3) that there are no extraneous conditions influencing the market value of power.

The problem of determining the proper size of a development is generally very complicated, and is influenced by many things outside the plant under consideration.

* *Electrical World*, Vol. 81, 1923, p. 219.

† *Proceedings*, Am. Soc. C. E., April, 1924, p. 450–452.

With the exception of chemical and metallurgical works, the cost of power is a comparatively small percentage of the expense of running an industrial enterprise. Such things as labor market, availability of materials, and transportation, have much more to do with where an industry locates than does the cost of power. This being the case, it may be said that, within reason, power is worth just what it costs to get it, no more and no less.

A hydro-electric site should be developed to the point where it is most efficient economically from the standpoint of the cost of power to the entire community within transmission distance. This involves a study of its relation in operation to other sites which will be developed in the future, and of the possibility of interconnection with other systems. The study of the power development program of the entire district within transmission distance may lead to a conclusion quite different from that where the study is merely to determine the most efficient combination of a given hydro-electric site with steam development. It is an economic mistake to take the very cheapest power in a way that damages or destroys possibilities for additional development at a slightly higher figure.

CHARLES B. HAWLEY,* M. AM. SOC. C. E.—Popular interest in hydro-electric development is widespread. Undoubtedly a part of this interest is due to the hope that water-power construction will soon make possible substantially lower prices for electric service, but this, of course, is a vain hope because the actual cost per kilowatt-hour at the switch-board, regardless of whether it is generated by steam or water, is always small in comparison with the cost of distributing this kilowatt-hour to the average consumer. For the most part, however, popular interest in water power comes from a realization that present fuel supplies are not without limit. Notwithstanding all the attention now being directed toward hydro-electric development, it is a fact that the aggregate capacity of all the steam-power plants being built throughout the country is several times that of all the water-power projects under actual construction.

Evidently there must be numerous reasons why this is so. Some of them have already been suggested in previous discussions; the speaker will review a few of these reasons very briefly, as they have a direct bearing on the present economic and political phases of this general subject.

First.—The preliminary negotiations in connection with the promotion of hydro-electric developments require many times as much effort and patience as in the case of a steam-power project. In the first place it is usually necessary to secure the permission of both State and Federal authorities. Thorough engineering surveys and investigations, including exploration of foundation conditions, must be made and reported on. The necessary lands and riparian rights should be secured, insofar as possible, in advance of construction. Property damages and interferences with existing roads, bridges, railroad rights of way, etc., must be considered. All these matters take time and patience, so much in fact that occasionally the promoters abandon the project temporarily in order to allow time to assist in working out their problems.

* Cons. Engr., Washington, D. C.

Second.—Another handicap to water-power development is the general belief that such construction involves greater hazards and uncertainties as to final cost and amount of power actually available than the steam plant. There is doubtless good reason for this belief. Severe floods may occur at inopportune times during the construction period; the exact cost of riparian rights and property damages can never be predicted in advance; and water-power plants are subject to a variation in annual output, due, of course, to the variation from year to year in the amount of rainfall and run-off.

Third.—The actual construction of a complete hydro-electric plant involving a dam, intake, conduits, power house, transmission lines and sub-stations usually requires a longer time than the erection of a steam plant of equal capacity.

Fourth.—The initial or capital cost of water-power development is inherently greater than that of steam-power plants. A larger amount of capital must therefore be raised.

Fifth.—The complete regulation of a stream is necessary before water power becomes independent of steam reserves.

Sixth.—There is a wide difference of opinion between the promoter and the banker on the one hand, and the public utility commissions on the other, as to a fair compensation for the effort required to promote and develop water power.

Recognizing these and other similar conditions as actual handicaps to water-power development, what then are the chief economic reasons for pursuing the construction of hydro-electric plants in districts where steam-power systems are already in control of the market?

In answer to this question there are perhaps only two important reasons. In the first place, water-power developments having ample storage reservoirs are peculiarly adapted to operation under low-load factors, whereas steam plants are most economical when carrying full loads continuously. As an explanation of this, it is known that when steam turbines are operated during peak-load periods only, heavy standby losses are incurred during the remainder of the day as boilers of corresponding capacity must be banked and held ready for service. On the other hand, hydro-electric units, when idle, are free from practically all standby charges except fixed charges; and in the matter of fixed charges it may be said that the capital cost per kilowatt of such additional capacity in a water-power plant such as would be required for peak-load service only, is usually about one-half the cost of peak-load capacity in a steam plant.

The other important incentive to hydro-electric development is the desire to pre-empt power sites against the time when the cost of fuels will be higher and the needs for power service much greater than they are to-day. The appropriation of water-power sites by steam-electric systems also tends to shut off the possibility of destructive competition.

The general demand for power in this country has been doubling every five or six years, and there is no present indication that this rate of increase will be lessened in the future; hence the necessity for a more complete utilization of water-power resources. Ten years ago hydro-electric securities were acceptable to only a few investors and only a limited number of bankers would take

any interest whatever in the promotion of such projects. As water-power development was considered hazardous, it was undertaken only by such companies as could be financed without appeal to the investing public. This limited the activity in hydro-electric construction to a comparatively small number of corporations, and perhaps indirectly stimulated the popular belief that the country's water-power resources, like its forests, should be withheld for a time from development—the reason for this belief being that otherwise all water powers would ultimately come under the control of the few corporations interested. This kind of conservation, however, is now obsolete. There is no longer any desire, or necessity for conserving water power by prohibiting its development, but instead it is the limited supply of coal, gas, and oil that should be conserved through the utilization of water power. What then can be done to promote and encourage hydro-electric development?

It is true that a few of the handicaps to water-power development are inherent and cannot be corrected. They can be offset, however, but this will require adequate legislation and perhaps slight changes in the policies now commonly in vogue among public service commissions throughout the country. The speaker hesitates to open this phase of the subject because doubtless many believe that there is already too much legislation involving the control of power development. Granting, however, that some sort of legislative control is the wish of the majority of the people, the following observations from personal experience are made for the consideration of those who are interested especially in this subject.

In the first place the Federal Power Law enacted in 1920 has done much to create beneficial interest in National water-power resources; it has facilitated power development on public lands and on navigable rivers, at the same time adequately protecting the interests of the general public. However, from the viewpoint of many who are willing to engage in water-power development, the law appears either unsatisfactory or inadequate in its definition of those streams which are to be subject to Federal control, for it handicaps by taxes and regulations, even though moderate, many new projects, in comparison with those older developments which were constructed before 1920 and which are therefore exempt from such taxes and regulations. Parts of the present law permit honest differences of opinion as to the actual intent of the lawmakers. Consequently differences of opinion exist with regard to the regulations now in force. This condition of uncertainty must be corrected either by amendment to the law or by precedent in its administration.

Another way by which water-power development can be encouraged is by correcting the present inadequate State legislation, especially with respect to rights of eminent domain and the methods of procedure under the rights conveyed by law to water-power companies. Here again precedent is lacking, and honest differences of legal opinion are possible, so that expensive and unjustifiable delays often result, even though the intent of the law is clear. In one particular instance where the dam proposed by a power company was going to flood out the pumping plant of a water-supply company, it was necessary for the power company to buy and undertake to operate the entire plant and distributing system of the water company merely to avoid the injunction and

delay that would have resulted if condemnation proceedings had been instituted to secure permission to reconstruct the pumping plant at a new site.

In another case where it was necessary for a power company to reconstruct a highway bridge at a higher elevation, it was found that the County Commissioners owned the structure; the County Court took jurisdiction as to the relocation of the bridge and highways; the State Highway Commission had jurisdiction over the changes in approaches to the bridge, as it was on a road designated as a State Highway; the State Water Supply Commission also had authority over all construction operations in or across streams; and the United States War Department likewise took jurisdiction, as the river was considered a navigable stream because at one time it had been used to float logs and rafts. Fortunately, while efforts were being made without success to bring about some agreement in the requirements of all these authorities, a new State law was passed setting up a commission which would have final power in such matters insofar as the State was concerned; and thereafter the necessary permission was promptly secured to raise the bridge. This, however, did not prevent all the other authorities from holding hearings, nor relieve the power company from attending such hearings and presenting evidence, although the procedure in each case was perhaps more or less perfunctory.

With respect to the taking of private property by condemnation procedure, the laws of some States permit the filing of a bond with the Court to cover all possible damages, and thereafter allow the immediate occupancy of the property in question. This is satisfactory. In other States, however, the petition to the Court must be advertised, subsequent appraisal made by jury, damages assessed by the Court, and payment actually made by the power company before the property can be occupied. As appeals are possible to superior Courts, the process is one which is subject to long delay, and the law is therefore of no real benefit to the power company, which must have the property in order to complete its construction work. Moreover a large number of such cases filed at one time with the ordinary County Court would completely clog the docket with the result that prompt action could not possibly be secured.

Another law conveying the right of eminent domain limits the cases which can be heard to those involving damages of more than $3 000. This gives the power company no recourse except to pay a recalcitrant land owner at least this amount for his property irrespective of its size or value. Some States are now enacting water-power laws patterned after the Federal Power Law; these can be made to embody provisions which will materially assist a water-power company in determining more rapidly and fairly the amount of its liability for property damage. Care should be taken, however, to avoid undue duplication either of authority, or of taxation, or of control over water-power development.

A third opportunity to favor water-power construction rests with the public service commissions. These commissions have done much in the past to stabilize public utilities. One of their general policies is to prevent destructive competition between utilities by allotting each particular district by charter or franchise to only one power company, and then to require as good service as

is practicable, allowing rates which will permit a fair return on invested capital. This policy is constructive and usually entirely satisfactory. It does not, however, encourage water-power development in that no premium is put on the economy that might result from lower generating costs. There is no intention to imply that the average power company is not interested in the efficiency of its steam turbines or in the cost of generating power, but the difference in cost of the cheapest water power as compared with the most efficiently generated steam power is so small a proportion of the retail price to the consumer that there is no real incentive urging the company to make the necessary effort to develop water power. If the desire to pre-empt water-power sites and to utilize hydro-electric power for peak-load service is not alone sufficient to bring about the development of all the better water powers, then, with a view to saving coal, it may some day be necessary for the public service commissions to insist that an electric utility occupying territory in which water-power possibilities exist shall develop these powers for the purpose of supplying their normal increases in demand rather than make further additions to their existing steam plants, allowing adequate rates, of course, to compensate the company.

Such a policy, practically worked out and administered, and supplemented by modifications in Federal and State laws, particularly with a view to facilitating the exercise of rights of eminent domain, would result in the prompt utilization of the nation's water-power resources and thereby the conservation of its fuels.

W. S. LEE,* M. AM. SOC. C. E.—The economics of hydro-electric development is very prominently before engineers of this country. Probably the next ten to twenty years will be the hydro-electric decades in America. People are thinking more about hydro-electric development, more about economics and conservation.

This paper, although covering a great many subjects, in harmony with its broad scope could not be devoted to a detailed discussion of these various topics.

There are three principal items to be considered in such a project: First, the site; second, the money; and third, the market. If one is missing the project will not be successful, or perhaps may not be constructed.

The variations of stream flow that prevail in Michigan, prevail in the Southern Appalachian slope to a much greater extent; there are variations of 200 000 sec-ft. for a drainage area of 1 400 sq. miles. It can readily be seen that to provide for this divergency is difficult and expensive. Hydro-electric development requires a co-ordination of the companies to smooth out these irregularities, an interconnection to assist one another in the event of a depression or a peak in their power supply. This does not necessarily mean that these companies must be one; there can be several companies, but there must be some plans of interconnection between them, in order for each to obtain the greatest amount of energy or the best value out of its plant. The Southeastern section has believed in interconnection for years. The speaker urged it in 1910 before the American Institute of Electrical Engineers. Years

* Chf. Engr., Southern Power Co., Charlotte, N. C.

ago it was found that these variations of stream flow in one section could be used to help out where the discharge was more uniform. The author's treatment of this phase of the subject is impressive.

He has discussed the use of water. When the water in the tail-race is turbulent, no matter who the designer was, nor what the type of hydraulic apparatus used, energy is being wasted. When the discharge flows in a quiescent state, with a reasonable velocity, an economical point in the design of the tail-race has been reached.

In the design of dams it is important to take care of the enormous overflow and, above all else, to prevent sliding, especially if large quantities of water must overflow and pound the structure.

The speaker is opposed to more legislation and does not see how the public can be hurt by the public utilities, such as power companies, if the rate-making power is left to the commission. Engineers who are interested in developing cheap power from oil and coal are standing on guard saying, "You can't put rates too high, or we will compete, using steam". There are many good laws on the statute books; among them is the Federal Water Power Act especially as about 75% of the undeveloped power of the United States is on Government property; but it is not a good law if it fails to cover the property that the United States owns or holds, and if the Government can turn every spring or brook into a navigable stream by calling it such. The Government owns Muscle Shoals *in toto*—not water rights—and it is proposed to give it to an individual for an individual use; on the other hand, if an individual wants to build a plant on some stream that floated a log back in 1850 he must go to the Federal Government for permission, and turn the plant back in 50 years, after furnishing all the money for development. Such laws shake one's faith in all this additional legislation.

Building a hydro-electric plant is not like building a steel office building, or a structure where there is no flood during construction. A well-trained organization can effect a saving of 25% in developing a hydro-electric plant by co-ordinating the design, construction and erection. There are many ways to do this and not only effect immense savings, but insure the safety of apparatus from flood while building and shorten the time of completion. The construction plans and erection plan should be arranged so as to escape the danger of the flood waters as quickly as possible, and thus save money.

BRENT S. DRANE,* M. AM. SOC. C. E.—Just two thoughts occur to the speaker; they are hardly technical but apply rather to the engineer's duty to himself and to the public as a citizen. Professor Mead has emphasized the dependence of the profession on some continuous agency for furnishing stream-flow data; and that must be, as all know, a governmental contribution to the professional need. He has mentioned the fact that the Oklahoma record, which would have been so valuable in the case he cited, was terminated in 1908, just before extremely significant data might have been secured. It is a costly coincidence that in 1908 a spirit of economy in the Federal Administration caused the abolition of the Water Resources Branch of the United States Geological

* Director, North Carolina Geological and Economic Survey, Chapel Hill, N. C.

Survey. The cost of this economy to the people of Oklahoma City has been vividly illustrated by Professor Mead.

The speaker is convinced that the public grows in intelligence year by year, and responds more and more to the thought of the engineer. The profession is under obligation to this public constantly to impress on it that it owes, through governmental agency, a fundamental duty in providing basic data for the use of technical experts who serve the public, thereby effecting public economy.

The other matter of importance was mentioned by Mr. Hawley in his admirable discussion.* A majority of engineers who deal with hydro-electric projects at all deal with relatively small independent plants. Do they not, as a profession, owe it to the public, to promote the conception of a power stream as a unit—the conception of the stream ideally developed as having a certain potential economic value which may be considered as a definite amount of capital wealth, and the attendant realization that anything less than a development of this kind means the destruction of capital wealth? Every one is familiar with the disastrous effect, when a relatively uneconomic development interferes with that of one or more preferable sites and with the complete attainment of the stream's potential value. Evidently the ideal attitude may involve sacrifice on the part of the individual engineer; but is it not true that the members of the profession as good citizens do owe just this duty to the public as part of their contribution to the conservation of National wealth?

* See p. 888.

RESEARCHES ON THE STRUCTURAL DESIGN OF HIGHWAYS BY THE UNITED STATES BUREAU OF PUBLIC ROADS

Discussion*

By Jacob Feld, Assoc. M. Am. Soc. C. E.

Jacob Feld,† Assoc. M. Am. Soc. C. E. (by letter).‡—The design of highways, like that of many other structures, has been brought to a more scientific basis by logical deductions from numerous large sized experiments, of which those reported by the author are probably the most far-reaching and the best planned. As in many other engineering designs, trouble in highways arises not in the structure itself, but in the design of the foundation. The infinity of possibilities as to the nature of soils and the large number of seemingly small, but really important, factors which enter into any investigation of materials not readily classed as solids or gases, makes any design in which forces or reactions from such materials have to be considered, indeterminate in many ways. According to the best present practice, the method of design, as for all truly indeterminate structures, is to find one possible solution of equilibrium compatible with the conditions of the problem and to trust to the Moseley Principle of Least Resistance that the true solution of equilibrium will result in stresses no larger than those found. To determine the worst (or end) conditions which may govern the design, full sized experiments are required in which the factors are varied from one extreme to the other.

No logical design can be expected until the materials are scientifically classed into groups containing those which act similarly under similar conditions. Great progress has been made in the classification and standardization of road materials in the past few years; the sub-grade has not received equal attention and is now the weak link in the chain of highway design.

The investigations of the Special Committee of the Society on the Bearing Value of Soils, etc., have proven the importance of the colloid content in the action of soils. This fact may be expressed in another way—that the absolute size of the soil particles is an important factor. (A material is called colloidal when its individual particles are from 0.08 mm. to molecular dimensions in size; the hydrogen molecule is approximately 0.00000016 mm.). Until quite recently, the relative grain size or gradation received major attention.

The importance of the absolute size of particles is evident from the list of tests of the U. S. Bureau of Public Roads to distinguish and define the various properties of sub-grade materials. The first test, mechanical analysis, determines the percentage of materials of each size. The remainder of the list

* Discussion of the paper by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., continued from May, 1924, *Proceedings*.

† Designing Engr., D. B. Steinman, New York, N. Y.

‡ Received by the Secretary, April 30, 1924.

reads like a description of the standard tests in physical chemistry to determine the properties of a colloidal substance. There is not much hope, however, in this type of analysis, for even a cursory study of colloid chemistry discloses the fact that, although there is a large mass of empirical data, there has been little scientific progress. In addition, chemists have naturally devoted their attention chiefly to the action of chemicals on colloids. A little has been done in the matter of temperature changes; but very few experimenters have investigated the effect of loads, impact, vibration, and similar factors. Further, practically all research in this field has dealt with colloids, *per se*, and not with materials containing a colloidal constituent. Throughout this work discrepancies and unexplained contradictions in results exist.

The very recent work in soils as an engineering material marks a departure from the customary research in materials of construction. All design methods at present are purely empirical or are based on data checking a definite theory. The first work in any field was an attempt at a complete solution of the problem—the effect of definite forces on the material under consideration. Later, the investigation was broadened into a study of the materials themselves. For example, the methods of designing metallic structures were known long before any investigation was begun into the nature of iron or non-ferrous metals and into the methods by which these metals could be controlled to give certain desirable characteristics. After a few experiments concerning the proper design of foundations, the distribution of loads through soils, etc., the research work is now practically entirely devoted to a study of the nature of soils. This trend will probably result in a more correct first solution, but meanwhile there is nothing tangible to guide the designer. For example, it is assumed that the soil reaction beneath a column footing is uniform. It is known that this is far from exact; yet, what is a more correct assumption?

Several points in the author's discussion of the sub-grade are of especial interest to the student of colloids. The importance of soil shrinkage is stressed. Can the abnormal shrinkage of the sub-grade of concrete roads be caused by the action on the colloids in the soil of the chemicals leached from the concrete by the excess water? Clay, for instance, is easily coagulated by lime. A control of the colloids should eliminate part, at least, of the soil shrinkage below pavements, just as the shrinkage of farm land is controlled by proper chemicals.

That part of the paper dealing with the influence of the bearing area on the bearing value of soils is the first available experimental data on this phase of the soil problem. In 1923, the writer investigated the literature regarding the proportionality of bearing area and supported load, and found practically nothing. To be exact, the only experiments on record were a few large tests on some building foundations, the New York Municipal Building especially, some small tests by Yankovsky, and a few incidental tests by Crosthwaite and others in connection with similar work. These references, with a theoretical paper by Keck, in which the proportionality of bearing area and pressure is experimentally proved for several solid materials and also claimed for sands and soils (although no experimental data are given) constituted the only literature that a rather careful search revealed. The brief summary given by the

author points to a most important report which, it is hoped, will soon be published in full.

The author mentioned the fact that the bearing value of soils, especially sands, is highest for a definite moisture content, usually a moist condition. This is associated with the fact that the density of soils is a maximum at some low moisture content, that is, a definite quantity of soil will occupy the least volume when it contains a definite small percentage of moisture. An increase or a decrease in moisture causes an increase in volume. In explanation, this is usually assumed to be the effect of the surface tension of the water films covering all soil particles, drawing these particles together. The proper quantity of water covers each particle with no excess. Additional water fills the pores and, as water resists compression, tends to resist the decrease of volume to a minimum. Too little water permits grains to touch each other, corner to corner, and not arranged in closest possible relation to each other.

The author is to be congratulated on his clear presentation of a wealth of empirical data covering so many features of highway design. The writer has discussed only that section of the paper dealing with the foundation in which he is especially interested. He hopes that other discussions will point out the valuable features of the remainder of the paper.

INCREASING THE CAPACITY OF EXISTING STREETS

Discussion*

By Messrs. E. P. Goodrich and John A. Miller, Jr.

E. P. Goodrich,† M. Am. Soc. C. E.—It has been interesting to have had the economic element brought into the traffic problem. Modern engineering is a question of economics, fully as much as of the physical creation of things for the benefit of man.

In somewhat the same way that Messrs. Hill and Crane have calculated the saving due to double streets,‡ estimates have been made of the present cost of congestion in various cities. Offsetting those congestion costs, the costs of improvements have been determined, thus putting the whole problem on an economic or financial basis. It has been estimated that Cincinnati loses \$100 000 per day due to congestion. Not all of this, however, if capitalized, would be a proper expenditure for removing the physical difficulty.

The best practice is to make use of existing roadways to the utmost. Working along that line a scheme was devised, some years ago, which might be described as the platoon system of operation. For example, imagine a company of soldiers marching northward on Fifth Avenue; following it is another company with a gap between, succeeded by a third platoon. A similar series marches down the street. Obviously, where the gaps meet, cross traffic can operate. By proper spacing, it has been found that a system of rectangular streets could be so co-ordinated or synchronized as to keep any particular vehicle moving continuously in any direction without stops, the groups of such vehicles interlocking at the cross streets.

It can be illustrated in another way. All are familiar with the time space diagram. Assume the vertical ordinates as distances along Fifth Avenue; at appropriate points are the cross streets; time is plotted horizontally. Therefore, a vehicle starting at a certain point and moving up the Avenue, would be shown graphically by a diagonal line. A subsequent movement of vehicles would be represented by a diagonal displaced to the right. Those coming down would be represented by lines sloping in the other direction. On any cross street, the same scheme could be operated, if properly arranged.

A somewhat similar scheme has been in operation in Los Angeles for some time, traffic in parallel streets being operated and stopped in a synchronized manner, so that in every alternate street, it moves alternately. A system of semaphores, somewhat like the block system mentioned by the author, is in operation. Such a system would increase the efficiency of the present Fifth Avenue operation by 100 per cent. The Los Angeles system is far from perfect but is a step in the right direction.

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* Discussion of th paper by Arthur S. Tuttle, M. Am. Soc. C. E., continued from May, 1924, *Proceedings*.

† Cons. Engr., New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., May, 1924, p. 713.

It may hearten Mr. Davison who has discussed the situation in Pittsburgh, Pa.,* to hear that in practically every State in the Union the number of automobiles is approaching the saturation point. Doubtless the traffic will correspond approximately with the ownership of automobiles. That has been proven in different districts where traffic observations have been made over long enough periods.

Mr. Bartholomew† put his finger on an important point when he mentioned the elimination of parking. Many times that question has been raised, but it is yet to be answered satisfactorily.

JOHN A. MILLER, JR.,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—this paper treats of a subject of great interest and importance at the present time and contains a number of good suggestions for the relief of traffic congestion. However, the removal of tracks from the streets and the substitution of buses for the electric railways, a plan which the author seems to favor, would greatly increase congestion instead of relieving it.

He asserts that the economic advantages of the surface car over the bus in street capacity used have been lost to a considerable extent in crowded sections of the city because of the slowing down of speed. Experience in other cities where large numbers of buses have been operated, does not confirm this assertion. On the contrary the superiority of the street car over the bus is most evident where congestion is the greatest.

In order to reduce the cost per passenger-mile of motor-bus operation to approximately that of electric-railway operation, bus development in the United States has been confined almost entirely to vehicles operated by one man. This places a restriction on its carrying capacity, which may vary from 30 to 50, depending on the degree of crowding which is permitted, but will not ordinarily exceed 50 as a maximum. Under the same conditions of loading, a street railway car can carry from 100 to 150 passengers. It is evident, therefore, that there would have to be three or four times as many units on the street to carry all the traffic by bus. The resulting congestion would be much worse than the present condition.

Extensive investigation made by the writer over a period of years developed the fact that in congested districts, with buses and cars operating side by side on the same streets, the bus was from 10 to 20% faster than the car. This superiority in speed resulted primarily from the fact that the bus was making no attempt to handle all the traffic, but was simply providing a sort of express service for the limited numbers of persons whom it could accommodate. Time was saved because the buses received capacity loads at or near the beginning of the route, and proceeded without making additional stops. From this it is clear that the greater speed of the bus is the direct result of its smaller carrying capacity.

The 10 to 20% superiority in speed, however, is not enough to offset the 200 to 300% increase in the number of units which would be required if

* *Proceedings*, Am. Soc. C. E., May, 1924, p. 723.

† *Loc. cit.*, p. 721.

‡ Asst. Editor, *Electric Railway Journal*, New York, N. Y.

§ Received by the Secretary, June 3, 1924.

buses were to carry the entire traffic. Although the use of double-deck buses would reduce somewhat the number of units needed, their greater carrying capacity would increase the number of stops required, and this, coupled with the delays incident to passengers climbing up-stairs and down-stairs, would reduce their speed to a rate somewhat slower than that of the street car. It appears, therefore, that the theory of lessening traffic congestion by the substitution of buses for street cars is not sound. Moreover, mass transportation by bus is unquestionably far more expensive than by rail.

In his discussion,* Mr. Corbett states that wheel traffic is gradually absorbing that formerly carried by rail, because more people are riding in motors and fewer on street cars and in subways. Statistics do not bear out this contention. The figures of the Transit Commission (Table 15) show more passengers being carried by the railways in New York every year, the number in 1923 being approximately 14% greater than in 1920.

TABLE 15.—REVENUE PASSENGERS CARRIED IN NEW YORK, N. Y.

Year.	Surface lines.	Rapid transit lines.	Total.
1920	941 420 788	1 331 915 745	2 273 336 533
1921	977 652 062	1 418 649 471	2 396 301 533
1922	1 052 968 891	1 438 239 919	2 491 208 810
1923	1 071 736 876	1 506 076 001	2 577 812 877

It is very easy to state, as Mr. Corbett does, that all rail traffic should go underground. However, in view of the constantly increasing traffic on all the railways, how can sufficient subways be built quickly enough to permit the removal of surface and elevated lines? No one has yet been able to suggest a method.

As neither the substitution of buses nor the building of more subways is a feasible method of handling the passengers now carried on the surface, railway tracks must remain in the streets. It would be possible, however, to increase the capacity of existing streets by speeding up the movement of traffic. The present slow speed of surface cars which the author uses as an argument against them, is to a considerable extent the result of conditions that can be remedied. If traffic were regulated so that the tracks were kept clear for the cars and not obstructed by motor and horse-drawn vehicles of every description, and if traffic officers were encouraged to make a greater effort to co-ordinate their signals with the movement of cars, a substantial increase in speed for all traffic would be possible with no expense to any one.

It would be very unfortunate if the public became imbued with the idea of tearing up the car tracks in New York before any adequate substitute has been found to handle the tremendous traffic. Perhaps such a step is not actually contemplated, but the frequent talk to this effect, whatever its purpose, is creating a feeling of doubt and uncertainty in the public mind which is working a great hardship on the railways and seriously interfering with their program of reconstruction and improvement.

* *Proceedings*, Am. Soc. C. E., May, 1924, p. 706.

IMHOFF TANKS—REASONS FOR DIFFERENCES IN BEHAVIOR

Discussion*

By Messrs. George T. Hammond, John F. Skinner, John R. Downes, and David A. Hartwell.

George T. Hammond,† M. Am. Soc. C. E.—The behavior of Imhoff tanks, or indeed of any other variety of tanks employed in sewage treatment, is a matter of much importance to sanitary engineers. It would not be easy to find one whose intimacy with this subject is more complete than the author's.

The principal functions of these tanks are to remove the suspended solids from sewage, and to digest the solids removed. The extent to which the mechanical, physical, biological and chemical agencies are concerned in the removal and reduction of suspended matter in sewage, constitutes a very extensive study, to which some of the ablest students have devoted much time and labor without exhausting its uncertainties.

In a previous discussion,‡ the author emphasized the fact that "the principal thing in which the sewage treatment plants in this country have not come up to a desired standard has been their operation. There have been rather few cases where design and construction, and even methods, have been such as to cause the plant to be justly termed a failure. There have, of course, been improvements from time to time. There will always continue to be improvements in methods and designs; but we seldom seem to secure satisfactory operation."

It is gratifying to learn that the plants described in this paper are carefully operated. This permits a fair comparison of results in tank performance such as is seldom possible.

The comparative dimensions, proportions and cubic capacities of the sedimentation and digestion compartments are of much interest. It is obvious that every Imhoff tank has a limited capacity, and that almost any tank might be expected to operate without nuisance within its capacity. But this is not as simple as it seems. What is tank capacity, and how may it be predicted? Unfortunately, tank capacity is not by any means a constant value throughout the year, and to answer such a question is not easy in advance of the installation, as there are many factors of doubtful value in the problem.

Under the circumstances, it would be wise to instruct the operator to find the capacity by test, regardless of any statement that may have been made to him, and, having satisfied himself of its capacity under varying conditions, to operate it within the limits of safety at all times.

* This discussion (of the paper by Harrison P. Eddy, M. Am. Soc. C. E., presented at the meeting of the Sanitary Engineering Division on January 17, 1924, and published in *Proceedings* for May, 1924), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Brooklyn, N. Y.

‡ *Proceedings*, Am. Soc. for Mun. Impvt., 1922, p. 202.

Designers have not always been able to forecast the tank capacity correctly—it would be remarkable if they could—and no Imhoff tank will operate with success if overloaded. As an instance of actual and expected capacity, the speaker recalls that the Imhoff tanks at Baltimore soon after they were put into service, operating on about half of the flow for which they were intended, appeared to be doing pretty good work, with no foaming; later, after the full load was applied, they foamed excessively and developed other troubles not observed in the previous instance. Under half load, the capacity of the digestive compartments was about 2.0 cu. ft. per capita.

The best way to manage a tank that proves to be too small is to work it only at its actual safe capacity, unless its design is so bad that it seems to have no safe capacity. The speaker has never seen a tank that would not operate with some success if the load was correctly determined and never exceeded.

The first requirement of an Imhoff tank is sufficient capacity for the greatest burden it may be called upon to carry. After this—and also of prime importance—are the details of design, and the correct proportioning of the parts. In every instance this should be considered in relation to the local conditions and service required, as well as to the prevailing climate, temperature changes, character of sewage, etc. If special studies of the local sewage are not made, as to volume of deposit, water content of sludge, etc., it will not be possible to forecast the probable tank capacity before the tank is in operation.

The author has covered details of design as related to the problems presented in his paper about as fully as such a study requires.

Concerning design, there is little or no criticism called for at this time, although faults in design may intensify other troubles in a tank. The designs exemplified in the Rochester and Fitchburg plants are generally accepted as standard by American engineers. The Plainfield installation has supplied experience that has been fruitful. The Schenectady plant was not built as designed.

The important difference in these plants is to be found in the comparative depths of the tanks, and in the cubic capacity per capita provided in the digestion compartments. Other things being equal, the speaker would expect that the tank having the greatest ratio of depth to capacity would give the most successful performance.

The author's treatment of the rate of sludge digestion, and the per capita sludge accumulation in Imhoff tanks at relatively low and high temperatures, is very suggestive of the causes that lead to the overloading of a digestion compartment. The increase in required capacity per capita from 1.83 to 2.04 cu. ft. with a decrease of only 5° in temperature, is significant as to what may be expected to happen if ample space is not provided in this compartment.

It is probably a safe presumption that temperature effects take place more slowly in a deep tank than in a shallow one of equal capacity, and are not so marked; and that in shallow tanks, such as those at Schenectady, the maximum effects of temperature changes are to be expected. Actual, rather than average, temperatures are of importance, as in shallow tanks these changes may occur rapidly, slowing down the biological processes by lowering, and increasing them

by raising, the temperature. These changes are doubtless more rapid in tanks that have a large surface exposure in relation to depth. Where the depth is relatively shallow, the sedimentation compartment reaches relatively deeper into the digestion compartment, and therefore transmits the temperature of its contents more quickly to the digesting sludge.

This view of the subject may explain the causes of foaming, especially in a shallow tank, a rapid increase in temperature causing a stimulation of the biological processes and a more rapid evolution of gases.

The importance of temperature studies in connection with tank performance is no doubt very great. It seems probable that a low temperature, in the cool portion of the year, followed by a short warm season, may result in an abnormally large accumulation of digestible solids while the temperature is unusually low, followed by a violently active digestion of sludge in the warm period. Winter foaming is seldom observed.

The phenomena which accompany foaming and frothing in Imhoff tanks are, however, so divergent that it must be admitted that many explanations are possible. Tanks will foam when no sludge is present; or when filled with sludge; or when much or little sludge is present. The suspended matters are so light in specific gravity, that any causes which give rise to a rapid gas formation may result in foaming, such as a sudden increase in temperature, or even a change in barometric pressure. The presence of grease in the tank and an acid reaction are conditions often noted when foaming occurs. But foaming takes place when the condition is not acid and there is but little grease present, and when all explanations seem insufficient. At times during the Brooklyn experiments, and without any apparent reason, the entire body of sludge in the digestion compartments of the different tanks would rise to the top, usually without foaming or frothing, remain for a day or two, and then slowly settle. On such occasions, gas discharge from the gas vents was at least of more than average activity and eructations of gas were noted. At such times there was considerable odor present, and when foaming occurred there was a very persistent odor. Churning the scum with a hoe in the gas vents, or breaking it up with a paddle, usually sufficed to release the entrained gas and permit the mass to settle. At no time was the foaming so serious that it could not be controlled, but while it lasted the smell was obnoxious. The sedimentation compartments were at no time involved and very seldom showed any scum. The "free-board" was only 15 in. and was never overtopped.

It may be added that the Brooklyn Imhoff tanks seldom caused odors, and when they did, if it was not due to foaming—noted only in very warm days in summer—it was probably caused by a septic condition of the sewage entering them. At one time during the experiments all of the tanks emitted odors, and it was discovered that the main sewer had become very foul from deposits. After the sewer was cleaned, these odors ceased.

The idea that sewers and disposal plants will run without causing trouble if left to themselves is erroneous as all know; but the benefit from cleaning the sewers is not sufficiently recognized.

An excessive formation of scum is usually not observed in sewage from a combined system, although frequently in sewage that is relatively fresh, espe-

cially if from a separate system. This problem at times constitutes one of the greatest difficulties of the plant operation and is often related to foaming. Most frequently the scum consists of unbroken sewage matter—paper, sticks, fresh vegetable remains, shreds of string, trade wastes, rags, hair, etc., and a considerable amount of fatty material—all of these constituents being intermingled and in a condition that resists rapid bacterial action. The various scraps from the making of home brews are strongly in evidence, and very resistant to bacterial attack.

The effect of scum is, first, mechanical, as it plugs up the gas vents and even reaches below the slot line. Its influence on the bio-chemical processes of the digestion compartment has not been sufficiently studied, but it would appear at times to be the cause of an acid condition. Under all circumstances it is a nuisance to the operator.

Where the sewage produces large quantities of scum, as at Plainfield, fine screens would afford the best possible assistance in the operation of the plant. The occurrence of excessive scum was not noted at Plainfield previous to the installation of the Imhoff tank. There were at first four septic tanks—two, 100 by 50 ft., built in 1902, and two, 200 by 50 ft., built in 1905. These tanks were covered and had a working depth of 6 ft. They could be operated in parallel or in series. In 1908, they were reported "to be working well, and there is not a very heavy scum on the surface."* These tanks were succeeded by the existing plant, which serves a population of about 40 000, in the three municipalities of Plainfield, North Plainfield, and Dunellen, N. J.

The Plainfield Imhoff plant is one of the earlier plants in this country, and some of the theories under which it was designed have become modified by experience there and elsewhere. The digestion compartment is divided into five chambers. The greater part of the sedimentation took place in the chamber under the inlet for the sewage, and three of the remaining chambers accumulated little sludge. As a remedy, the rate of flow was greatly increased, so that all of the chambers might be useful, but this led to troublesome scum formation and foaming. Next, a fine screen was used, and this considerably reduced the formation of scum, and with some other changes greatly improved the operation.

If at first all of the sludge digestion chambers had been combined into one compartment throughout the length of the tank, it seems probable that much of the trouble would have been avoided. The depth of the digestion compartment and the length of the sedimentation compartment should prove sufficient for good operation, although the depth might have been greater with advantage and the sludge digestion capacity is apparently deficient.

The Schenectady plant is notable for its shallow depth—on account of construction difficulties, the original design, 21.33 ft. deep, was changed to 13.42 ft. It was put into operation in January, and the sludge had not ripened when the first withdrawal was made in the following June, so that it was very offensive. The addition of lime was apparently of little use; foaming began and persisted in flowing over the wall of the gas vents into the sedimentation compartment.

* Thirty-second Annual Rept., New Jersey State Board of Health, 1908, p. 408.

It is to be noted that the raising of the walls of the gas vents and the free interconnection of the chambers constituting the digestion compartment have been effective in greatly improving the conditions. With the occasional use of lime and with other expedients the tank now works.

In this case the faults of the Plainfield plant appear to be intensified. Its failure to function seems to have arisen from much the same causes, with the added difficulty of very shallow depth. It is possible that this also would have been a favorable condition for the fine screening of the influent.

In contrast the operation of the Rochester and Fitchburg tanks is strikingly different; yet in these latter plants functional disturbance may be observed and even occasional foaming (at Fitchburg). These difficulties appear to be caused chiefly by (1) the changing temperature of the sewage, and (2) its effects on the bio-chemical activity in the various compartments.

Unfortunately, so little is known at present of what actually takes place in the various compartments of an Imhoff tank that any picture one may draw is wholly inadequate and largely futile.

From an examination of the four plants described in this paper the following comments seem obvious:

(*a*) Depth of tank is an advantage. It not only improves the storage conditions and the concentration of ripe sludge at the sludge pipe outlet, but it also makes for more uniform temperature and slower changes, especially in the contents of the digestion compartment.

(*b*) Shallow tanks cover a relatively larger area, and are more subject to rapid changes in temperature. Sludge storage conditions in them are not so favorable as in deeper tanks; and the relatively larger area of the top surface, even if partially covered, affords a greater opportunity for scum formation, and quite possibly for acid conditions derived from the scum.

(*c*) It is apparent, and quite in line with the opinion of many sanitary engineers, that there should be not less than 2 cu. ft. of effective capacity per capita in the digestion compartment. Indeed, the tendency to increase this provision appears to be logical. Ample capacity affords an assurance of safety in operation.

(*d*) The removal of the finer, or colloidal, matters in suspension is advantageous, and may be accomplished by detention periods that, although relatively long, are not accompanied by any bad effects in a well proportioned tank; but detention for periods longer than three hours may be unfavorable. In some tanks, a shorter period than three hours is advisable.

JOHN F. SKINNER,* M. AM. SOC. C. E.—The author has brought to the consideration of this subject so much thought, experience, wisdom, and imagination, that he seems to have touched on all the reasons suggested for the difficulties in the operation of Imhoff tanks. His selection of four well known plants that have been visited by many sanitary engineers and municipal officials, and represent such different conditions, has been very happy even if the presence of so many variables makes a solution somewhat indeterminate.

* Cons. Engr.; Deputy City Engr., Rochester, N. Y.

In September, 1912, the speaker was privileged to take part in a two-day conference with Dr. Imhoff, the late Emil Kuichling, M. Am. Soc. C. E., and Edwin A. Fisher, M. Am. Soc. C. E., then City Engineer of Rochester, N. Y., at which time the difference between conditions in the Emscher District, Germany, and in northern American cities was most intimately discussed, with especial reference to Rochester.

Since then, the speaker has designed three more plants with Imhoff tanks, that have been constructed at Rochester, namely, the Brighton, Charlotte, and University Plants. The Irondequoit, or main Rochester Plant, referred to by the author* was designed by the late Mr. Kuichling; to his forethought is chiefly due its success, although the half of the plant first built was designed for a population of only 200 000 and is now operating with a 30% overload.

This discussion will be devoted mainly to the Rochester plants with such comparisons as suggest themselves and general conclusions therefrom.

Character of Sewage

All the Rochester plants receive typical domestic sewage with no preponderating trade wastes. The water, as stated by the author, is slightly softer than that at Plainfield, N. J.

The Irondequoit Plant receives combined sewage, the Charlotte Plant also receives some storm water. The Brighton Plant serves a sanitary district only, and the University Plant (just constructed) will receive sanitary and hospital sewage.

Freshness of Sewage.—The raw sewage received at the Irondequoit Plant is stale and often in a septic condition when it reaches the plant. Sewage reaches the Brighton Plant within an hour. There is more colloidal matter in the Charlotte sewage than at the other plants, and more odor, possibly due to the presence of the products of solids that have deposited during dry weather in some of the larger combined sewers which are still connected with the system.

Hardness of Water.—The Rochester and Lake Ontario Water Company supplies water to the Brighton and Charlotte territories, its hardness being 122 parts per million as compared with 65 for the Hemlock System which supplies most of the city.

Both kinds of water are supplied to the two Rochester plants (Irondequoit and Charlotte) that develop quantities of scum. The sewage of one is fresh and that of the other stale; the only common factor is that they both receive street wash, which seems to be the source of most of the scum. The hardness of the water has an effect, however, in that grease balls are found in the scum at Brighton and at Charlotte, as at Irondequoit. The action of grease will be discussed subsequently.

Preliminary Treatment

All sewage entering the Irondequoit Plant passes through racks with 3-in. clear openings, then through detritus tanks, 90 ft. long, at a velocity of about 1 ft. per sec., and then through Reinsch-Würl screens with ⅛-in. by 2-in. slots.

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* *Proceedings*, Am. Soc. C. E., May, 1924, p. 616.

By the time the screen plates are discarded the slots have been worn somewhat wider.

At Brighton grit chambers are provided for emergency, followed by flat, inclined, fine racks with 1/8-in. clear openings.

At Charlotte there are detritus tanks 40 ft. long, in which a velocity of about 1 ft. per sec. is maintained in shallow channels with V-shaped bottoms. Fine racks are used similar to those at Brighton, but more nearly vertical in position.

With sewage as old as at Irondequoit, fine screens are useful, as the material removed is mainly garbage and other relatively non-putrescible substances. The screenings would add 4 to 6 cu. ft. per 1 000 000 gal. of relatively inert matter to the contents, considerable of which would have to be skimmed from the settling chambers. Feces are largely comminuted before reaching the plant and pass through the screens. At Baltimore, Md., on the contrary, with equally old sewage, great masses of unbroken feces are raked from the racks. This is one of the many features which indicate that each situation must be studied individually in the preparation of a satisfactory design. With the fresh sewage at Brighton and Charlotte, feces are not so well comminuted. The fine racks collect rags, fruit rinds, and paper pulp, while feces are broken up at intervals with a hose stream and sent on to the tanks.

Subsequent Treatment

The designer of a tank must consider the subsequent disposition of the effluent. The Irondequoit Plant discharges into Lake Ontario 7 000 ft. from shore in water 50 ft. deep. The winds, currents, and temperatures of the lake and sewage are such that there is no shore contamination. A careful preliminary study of the conditions was made by George C. Whipple, M. Am. Soc. C. E., in 1913, and his work was repeated by Mr. M. J. Blew in 1921, after the plant had been functioning for four years. The findings of Professor Whipple were confirmed in every respect. The design of the plant called for a detention period of 80 min.

The Charlotte Plant discharges into the non-potable water of the Genesee River where ample dilution is provided. As the population is so variable, unusual conditions had to be met in the design, namely, the operation of the plant at a minimum discharge of 30 cu. ft. per min. and a maximum of 180. The territory includes a summer colony, a municipal park, and a bathing beach having a transient population. The detention period, therefore, is from 1 to 6 hours.

The effluent at the Brighton Tank, like that at Schenectady, N. Y., Plainfield, and Fitchburg, Mass., is sprayed on trickling filters. A detention period of at least 2 hours was provided for; it is about 3 hours at present.

The University Plant is similar in design to the Charlotte Plant, only smaller. It will discharge into the non-potable waters of the Barge Canal. It is constructed as a temporary plant and will serve a territory of 300 acres, including the Medical School of the University of Rochester now under construction. At some future time, as the city develops, this section will become

a part of a larger territory tributary to a plant to be constructed about five miles to the east.

Typical cross-sections of the tanks listed in Table 16 are shown in Fig. 4.

TABLE 16.—CHARACTERISTICS OF IMHOFF TANKS AT ROCHESTER, N. Y.

Plant.	Irondequoit.	Brighton.	Charlotte.	University.
Designed for population of............	200 000	15 000	10 000	3 000
Tributary population............	260 000	10 000	4 000	0
CHARACTOR OF SEWAGE:				
Flow, in million gallons daily........	32	1.2	0.33	0
Separate or combined............	Combined	Separate	Combined	Separate
Suspended solids, in parts per million............	163	106	390	....
Freshness............	Stale	Fresh	Fresh	Fresh
Industrial wastes............	Small amount	Small amount	Small amount	Hospital
Hardness of water supply, in parts per million............	65	122	122	65
Temperature, in degrees Fahrenheit	54	53	54	
PRELIMINARY TREATMENT:				
Screening............	Fine screens	Fine racks	Fine racks	Fine racks
Number of units............	10	1	1	1
Grit chambers............	Yes	Yes	Yes	None
Skimming tank............	No	Yes	Yes	Yes
SEDIMENTATION TANKS:				
Sedimentation period, in hours.....	1.1	3	6	2
Total depth of tanks............	33 ft. 10 in.	34 ft	30 ft. 6 in.	25 ft. 2 in.
Depth to slots............	11 ft. 4 in.	{ 13 ft. 8 in. } { 19 ft. 0 in. }	15 ft. 2 in.	9 ft. 8 in.
Effective length of flow chambers..	110 ft.	60 ft.	42 ft. 9 in.	26 ft. 0 in.
Width of flow chamber............	10 ft.	24 ft.	13 ft. 9 in.	9 ft. 6 in.
Number of flow chambers............	2	1	2	2
SLUDGE COMPARTMENT:				
Depth below plane of slots............	22 ft. 6 in.	15 ft. 0 in.	15 ft. 4 in.	15 ft. 6 in.
Depth below 18-in., neutral zone....	21 ft. 0 in.	13 ft. 6 in.	13 ft. 10 in.	14 ft. 0 in.
Depth of sludge hopper............	8 ft. 0 in.	5 ft.	5 ft. 6 in.	5 ft. 6 in.
Cubic feet per capita............	2.4	2.18	2.88	1.47
Number of hoppers............	3	2	1	1
Ease of communication............	Good	None	....	....
Solids, in cubic feet per capita per year............	0.321	0.236	0.214	....
GAS VENTS:				
Percentage of tank area............	26.8	12.9	6.75	5.12
Area, in square feet per cubic foot of sludge per capita............	0.0133	0.0085	0.0061	0.0064
Least width of vent............	2 ft. 6 in.	2 ft. 2 in.	9 ft. 6 in.	6 ft. 0 in.
Horizontal travel of bubbles........	0 to 6 in.	0 to 12 ft. 10 in.	0 to 13 ft. 3 in	0 to 9 ft.
Date first operated............	March, 1917.	March 1, 1916	October. 1921	....

FOAMING

Foaming has never occurred at the Brighton Plant or at the Charlotte Plant and only once, five years ago, in one of the ten Irondequoit Tanks. This was immediately corrected by drawing sludge. It was distinctly a matter of improper operation and occurred when the man in responsible charge was serving in the Army.

SCUM

Considerable scum forms at the Charlotte and Irondequoit Plants, consisting of leaves, matches, rubber, and grease balls. It is usually inoffensive and is removed when it reaches a depth of 1 ft. or more. Occasionally, sludge appears in the scum; this is hosed and paddled and thus driven down. Gas

action is still (January, 1924), noticeable at the plants, as the season has been quite warm. The presence in the scum of matches, which are not often seen floating in the settling chamber, has caused speculation. It has been suggested that they come into the tank smeared and weighted down with other material and float after being disentangled; on the other hand, they may have been water-logged and later floated by the formation of gas within them.

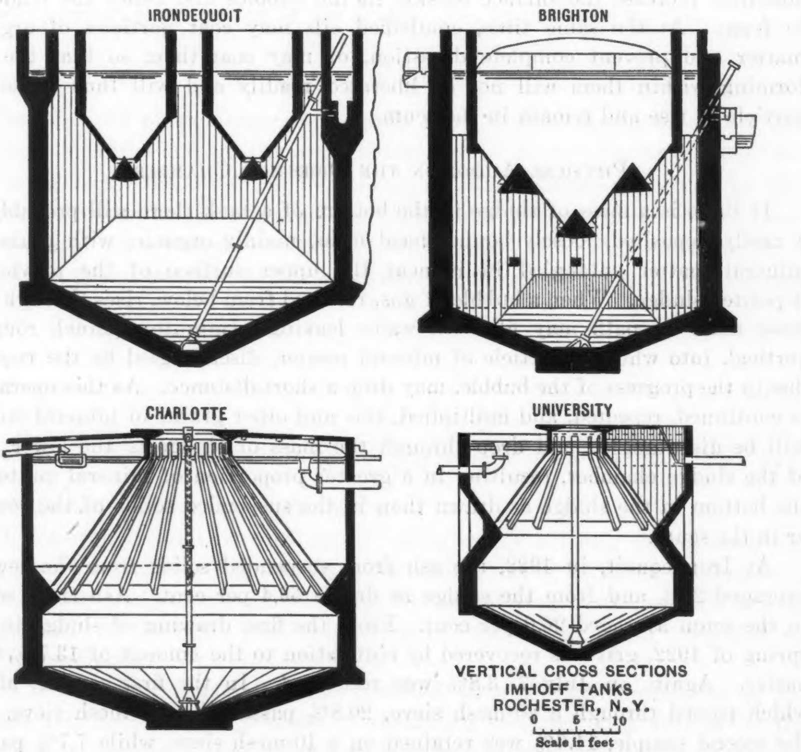


FIG. 4.

Little scum has been observed at the Brighton Plant and then only in detached masses, although the gas action is very free. This tank is unique in design in that it has only one settling chamber with four bottom slots, two gas vents, and two separate sludge compartments.

SLUDGE

The sludge from all the plants is well digested and inoffensive. It does not attract flies at any time and readily dries on the beds. It has contained as little as 79% of moisture and as much as 95%; it is dryest at the first drawing in the spring and less dense later. There is sufficient space in the tanks so that a drawing of well digested sludge can always be made at the advent of drying weather in the spring.

Grease

Insoluble soaps, formed by the substitution of calcium and magnesium in hard waters for the more loosely combined potassium and sodium in soaps, have been suggested as formers of scum. As found at Rochester, these grease balls are pellets 1 to 10 mm. in diameter. They do not coalesce and, therefore, do not cause foaming. Glycerine and saponifiable oils in solution will doubtless increase the surface tension in the bubbles and hence the tendency to foam. At the same time, emulsified oils may coat particles of organic matter and prevent complete digestion, or may coat them so that the gas forming within them will not be liberated readily and will thus cause the particle to rise and remain in the scum.

Physical Action in the Digestion Chamber

If there is a mass of sludge in the bottom of a tank there will probably be a newly deposited, loosely bound, fecal mass, mainly organic, with grains of mineral matter entangled in it near the upper surface of the previously deposited sludge. When a bubble of gas, released from below, rises through this loose mass its path may fill with water leaving a minute channel, roughly vertical, into which a particle of mineral matter, disentangled by the rupture due to the progress of the bubble, may drop a short distance. As this operation is continued, repeated, and multiplied, this and other grains of mineral matter will be disentangled and drop through the mass of sludge to the lower part of the sludge chamber, resulting in a greater proportion of mineral matter at the bottom in the sludge as drawn than in the suspended solids of the sewage or in the scum.

At Irondequoit, in 1922, the ash from suspended solids from the sewage averaged 28% and from the sludge as drawn 56.4 per cent. Ash from solids in the scum averaged 26.6 per cent. From the first drawing of sludge in the spring of 1922, grit was recovered by elutriation to the amount of 13.7% (dry basis). Again, on July 7, 3.8% was recovered. In the first sample, all of which passed through a 20-mesh sieve, 20.8% passed the 100-mesh sieve. In the second sample, 11.6% was retained on a 10-mesh sieve, while 7.7% passed the 100-mesh and 1.9% the 200-mesh sieve. A sample of scum was examined and after elutriation and decantation showed only 0.5% of grit (dry basis).

In general, therefore, the free mineral matter and the inorganic matter are greatest at the bottom of the sludge chamber and decrease toward the top.

The effect of difference in temperature is noticeable. In November, 1922, the temperature of the Irondequoit sewage had reduced to 54° Fahr., while it was 56° in the body of the sludge. When the temperature of the sewage becomes lower than that of the sludge, digestive activity is checked in the cooler zone at the surface of the sludge mass where light matter—mostly organic—is deposited. This material may lie relatively quiescent on the surface, for there is little mineral matter from street wash at this season of the year to weight it down. On the return of warm weather, this finely divided and readily digested material, agitated by rising bubbles from below, will digest with increasing rapidity and, if the surface of the sludge is too high.

will show itself in the slots at the bottom of the settling chamber, and, after these are sealed, by rising in the gas vents. This sequence of events can be averted to some extent by agitating the sludge during the winter so that the freshly deposited suspended solids may be distributed throughout the sludge mass and thus be partly digested during the cold period. This will have two beneficial effects—the reduction of quantity and the spring level of the sludge due to liquefaction and gassification, and the diminution of the violence of digestive action near the top of the sludge mass in the early summer.

Causes of Foaming

Consider the phenomenon of foaming from the standpoint of an air lift. As gas is liberated and rises through the sewage in the scum compartment (the slot being free), the condition to produce overflow of the vent would be that the higher column in the scum and gas chamber must equal in weight a column of sewage in the settling chamber. In other words, if h_1 represents the depth of sewage to the slot and h_2 the free-board of the gas-vent walls, the average density of the rising liquid will be $\frac{h_1}{(h_1 + h_2)}$ and the ratio of gas bubbles to volume in the scum and gas chamber will be $\frac{h_2}{(h_1 + h_2)}$. This condition can hardly be imagined; it would amount to $\frac{1.5}{(11 + 1.5)} = 12\%$ of gas at Plainfield; 11% at Schenectady; and 8.4% at Fitchburg.

If, however, the slot is sealed by the rising sludge not only may a condition exist more favorable to foaming from the standpoint of balanced columns but, as the rising sludge level contracts the horizontal area of the scum chamber, the gas will rise through a smaller quantity of liquid, more readily approaching air-lift conditions.

If, in addition, a mass of scum is added, with the surface tension of the bubbles increased by the presence of saponified material, and, further, if the gas vent is so narrow that scum and bubbles coalesce and bridge across the opening without breaking, conditions are ripe for foaming.

In the speaker's opinion, the causes of foaming at the various plants considered by the author are, as follows:

Schenectady.—Design.—

(*a*) Shallow tanks, eight in line, may be only about three-eighths efficient in digestion and release of gas, hence the loading may be eight-thirds as great as indicated by actual capacities.

(*b*) Small sludge compartments made smaller in effect by poor intercommunication.

(*c*) Trough bottoms do not lend themselves as well to sludge drawing as hoppers.

(*d*) Narrow gas vents.

Plainfield.—Design.

(*a*) Five tanks in line may be only three-fifths efficient in digestion and release of gas, hence the loading may be five-thirds as great as that indicated by actual capacities.

(*b*) Small sludge compartments made smaller in effect by poor intercommunication.

(*c*) Narrow gas vents.

Fitchburg.—Design.—

(*a*) Tall, narrow, scum compartments and gas vents favor air-lift action.

Fitchburg.—Operation.—

(*a*) Sludge carried too high.

Rochester.—Operation.—

(*a*) Sludge carried too high.

DETERMINATION OF SLUDGE CAPACITY.

The author suggests that "cubic feet per capita" is not the soundest basis for designing the capacity of an Imhoff sludge compartment. This, again, is another way of saying that each situation must be studied individually.

Attention is called, however, to the comparison of the four plants shown in Table 17.

TABLE 17.—THEORETICAL COMPARISON OF SLUDGE ACCUMULATION

Plant.	Sewage flow, in gallons per capita per day.	Suspended Solids.		Sludge, 90% water, in cubic feet per capita per month.	4 Months' Collection, in Cubic Feet per Capita.		Quantity After 25% Settlement and Digestion, in Cubic Feet.		Dr. Imhoff's rule, in cubic feet per capita.
		Parts per million.	Cubic feet per capita per month.		50% removal.	75% removal.	50% removal.	75% removal.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Schenectady......	92	163	0.060	0.60	1.20	1.80	0.90	1.35	....
Plainfield.........	85	166	0.056	0.56	1.12	1.68	0.84	1.26	....
Mean, Sanitary...	....	....	0.058	0.58	1.16	1.74	0.87	1.30	1.20
Fitchburg	89	219	0.078	0.78	1.56	2.34	1.17	1.76	....
Rochester	123	163	0.080	0.80	1.60	2.40	1.20	1.80	....
Mean, Combined..	....	....	0.079	0.79	1.58	2.37	1.185	1.78	1.80

It is obvious that by whatever method the proper capacity for the sludge chamber is found, it is directly proportional to the population and therefore, ultimately, must be on a per capita basis. It is equally obvious that it is

directly proportional to the quantity of sewage per head per day, to the suspended solids and the proportion of them removed by sedimentation, and to the number of days in the period when sludge cannot be drawn, due to weather conditions. It is also inversely proportional to the solid content of the sludge and is reduced by the amount of digestion during the cold period.

In determining the capacities of various parts of a tank, one should first, for the particular set of conditions, design the sedimentation chamber for a desired percentage removal of suspended solids. Having assumed this quantity and the total per capita suspended solids in the sewage and the length of period during which sludge cannot be drawn from the tank, a general formula may be evolved, as follows, assuming sludge with 90% of water in the tank, and 25% reduction by settlement and digestion during the cold period:

$$Q = \frac{D\,G\,R\,S \times 10}{1\,000\,000 \times 7.5 \text{ (gal. per cu. ft.)}} \times 0.75 = \frac{D\,G\,R\,S}{1\,000\,000}$$

in which

D = Period, in days, when sludge cannot be drawn;
G = Gallons of sewage per head per day;
R = Portion of suspended solids removed, expressed decimally;
S = Suspended solids in sewage in parts per million;
Q = The sludge space required, in cubic feet per capita.

In Table 17, Column (5), there is an assumption which may be varied to suit given conditions of temperature, etc., affecting the rapidity of digestion. The same is true of the 25% reduction assumed in Columns (8) and (9).

This equation is submitted as a convenient and flexible method of reaching a reasonable solution of an important problem. Like all determinations involving variables of unknown definite value, a reasonable factor of safety is to be commended as well as a careful comparison with existing plants.

Aids to Proper Functioning

It is evident that the most careful operation cannot prevent foaming where the design is at fault or where the plant is overloaded, likewise that careless operation may result in difficulties with the best designed plant. It would seem that the prime requisite is a liberal and well balanced design and one which considers ease of operation.

The following points are suggested as important:

Design.—

(*a*) Gas vents: Wide, but not necessarily of large area.

(*b*) Scum chambers: Ample in width and connecting freely with the gas vents.

(*c*) Sludge compartments: Ample in capacity and not more than three in line, in which case the intercommunication must be easy. Ample means of agitating the sludge with water jets, should be furnished.

(*d*) Settling chambers: Design with relatively large area and shallow overflow, and with provision for reversal of flow in the case of more than one sludge compartment.

(*e*) Preliminary treatment: Provide fine screens or fine racks and skimming chambers or deep scum-boards, so that large objects, floating matter, and scum may be kept out of the tank.

Operation.—

(*a*) Routine: Keep screens cleaned and floating matter skimmed. It is advisable to drive down undigested material in the gas vents and to remove the digested scum when it reaches 1 ft. in thickness. Agitation of the sludge some time before drawing and during the winter, if the sludge capacity is not large, will prove effective. A regular reversal of the flow—fortnightly or monthly —depending on the relative depths of sludge in the different compartments, is essential.

(*b*) Records: Keep records of temperatures and quantities and all operations both physical and laboratory, also, a journal, recording those conditions and occurrences not readily tabulated.

JOHN R. DOWNES,* ESQ.—The author is to be sincerely thanked for bringing together the available information on Imhoff tanks and focusing it on four installations of known operating characteristics.

The speaker approves Mr. Eddy's statement as to the two primary functions of Imhoff tanks, namely, the deposition of solids in the upper compartment and their digestion in the lower compartment. Many persons seem unable to rid their minds of the idea that the passage through the tank does something to the liquid other than to separate it from the solids. Engineers should disabuse their minds of this relic of septic tank lore. The experience at Plainfield, N. J., checks nearly all of the author's conclusions except as follows.

Although it is strictly true that the degree of removal of suspended solids depends primarily on the length of the detention period (with due attention to velocities) this has little application within practical limits in the case of the Plainfield sewage. In 1912, Weston Gavett, formerly Acting Chemist in Charge at the old Plainfield plant, showed that practically all sedimentation occurred within 50 to 60 min. after entrance. Although his work was with quiescent settling in the laboratory, it practically checks the actual experiences of operation.

The speaker disagrees as to the digestion of the minute solids† and believes it would be better to state that the more finely divided solids which are removed to a greater degree by a long detention period are in a state of division more accessible to the disintegrating organisms, but the success of the attack depends on the composition of the solids in question. At Plainfield, these finely divided solids are largely composed of grease and are digested with the greatest difficulty. This point is covered in a way by the statement as to the effect of excess soap grease at Plainfield and at Schenectady, N. Y.

* Superv. Engr., Plainfield, North Plainfield and Dunellen, N. J., Joint Sewage Disposal Works, Bound Brook, N. J.

† *Proceedings*, Am. Soc. C. E., May 1924, p. 617.

The author states that at Schenectady it was possible by building up the walls of the gas vents to cause the foam to pass over into the sludge pipe. At Plainfield, this was not successful as the foam broke in passing over the edge, drying instantly and refusing to run down the pipe.

An imitation of the dense, ripe, quick-drying sludge produced naturally at Rochester, N. Y., and Fitchburg, Mass., can be produced from the thin, watery sludges obtained at Plainfield and Schenectady by the addition of a small quantity of alum. Observation of the transition of the sludge from the lighter to the denser state suggests that time alone would accomplish the change if the size of the digestion chamber were great enough to permit digestion to proceed to its ultimate end. Whether this change is due to the continued action of the same groups of organisms, to the substitution of other organisms for them, or to the reaction of the end products of one group on those of the other group, is not now clear. Either of the latter two views would help to explain the necessity for the maintenance of a preponderance of the ripe sludge in the tank in order to keep control of the digestive process. The difference in capacity of the digestion chambers at Rochester and Fitchburg on the one hand and at Schenectady and Plainfield on the other, is sufficient to allow for this time factor and, therefore, to explain largely the difference in action.

The author states* that there are approximately 55 grammes per capita of suspended solids at the two plants treating domestic sewage and 75 grammes per capita at the two handling combined sewage. The extra 20 grammes, which is ascribed to street wash and is probably of a gritty, mineral, nature, may well be of actual service as a dilutent of the slimy, greasy mass of solids. A small proportion of sand in the solids may render them more penetrable to the attacking organisms, just as a small portion of sand in a dense clay will render it more permeable to water. The slight migration of such mineral solids due to gas movement in the mass may even produce an attrition effect. The solids from street washings in this day of paved, horseless streets must be more of a mineral nature and less of a burden on digestion capacity than was the case when the conception of the effect of such solids was established twenty years or more ago. Such solids, although represented by a weight of 20 grammes in 75, comprise only about 1% by volume of the freshly settled solids.

In computing Table 8† the author assumes that the sludge will contain 15% of solids at the bottom of the tank, and 10% at the surface. It is necessary to assume some such figure as a starting point; but this is the very assumption which seems to have created all the disturbance at the Plainfield plant. The designer has told the speaker that he calculated the sludge storage capacity on the basis of the results obtained at Fitchburg where sludge containing 12 to 15% of solids is common. The author has pointed out, on the other hand, that the conditions at Plainfield result in a sludge of about 7.5% of solids; actually, 6% would be more nearly the average.

* *Proceedings*, Am. Soc. C. E., May, 1924, p. 624.

† *Loc. cit.*, p. 630.

The speaker agrees with all the author's six points stated in Conclusions 25 (*a*) to (*f*),* as to the essentially unfavorable conditions at Plainfield and Schenectady.

The New Jersey Sewage Experiment Station has under way several of the investigations mentioned by the author, including temperature, hydrogen-ion concentration, quantity and composition of gases, and kind and function of organisms. It is not possible to study tanks of different depths at Plainfield, but different depths of sludge compartment will be compared.

The first trouble was due to acid-foaming with attendant odors. The later foaming has been alkaline, with little odor. Foaming always occurs when the material is near neutral or is acid. It was a question whether there was a lack of sludge digestion space or of sludge drying space. The use of alum solved this problem, by hastening the time of drying of the sludge so as to permit its removal from the tanks as fast as it was ready. The trouble is that there is not enough space to maintain the necessary preponderance of old sludge and thus control the digestion.

The difficulties encountered at Plainfield have been:

1.—The excessive formation of scum resulting in (*a*) odors, and (*b*) excessive manual labor to remove it.

2.—Return of solids through the slots to the sedimentation compartment and the consequent fouling of the liquid; and, again, an excessive labor demand for cleaning the surface.

All these difficulties may be said to be due to the fact that the solids neither stay at the bottom to form sludge nor rise to the surface to release the entrained gases and re-settle by gravity. A considerable portion of the solids has a specific gravity less than unity, being of a fatty nature, and, after being released from the entangling heavier solids which originally dragged them down into the lower compartment, remains permanently at the surface. Worse than this, however, the gases which buoy up the remainder of the solids are in the form of minute bubbles, mere pin-points, such as give the milky appearance to air-filled tap water. True, the minute bubbles escape from the tap water in time; but in the Imhoff tank the buoyancy of these small bubbles is exactly counterbalanced by the weight of the solids in which they are entrained and held as individual bubbles, so that gases and solids hang suspended together in an almost jelly-like mixture with the liquid, filling every part of the digestion compartment.

In other plants where the gas forms in bubbles of sufficient size to carry solids to the surface with some momentum, there may be a neutral zone about the slot and the trapped slot becomes effective in preventing the return of solids by diverting the course of the rising gas and solids. Where the mixture hangs suspended, however, as at Plainfield, there is no momentum; consequently, there can be no diversion, and the trapped slot becomes useless. The solids thus suspended may amount at times to as much as 3 per cent. Interchange between the compartments by diffusion, thermal currents, and displacement cannot be prevented.

* *Proceedings*, Am. Soc. C. E., May, 1924, p. 644.

At first glance the difference between the cross-sections of the Plainfield and Fitchburg plants seems to lie mainly in the communicating opening between the hoppers. A study of the different volumes resulting from the two kinds of hopper treatment is interesting. The author states that the difference in depth below the slot is 31% (Fitchburg, 9.5 ft., Plainfield, 7.25 ft.). The effect of the Fitchburg hopper, however, is to concentrate the heavier, older sludge from an area of 900 sq. ft. in approximately the same area as that filled from 150 sq. ft. in the Plainfield hopper. In Fig. 5 one hopper is shown superimposed on the other. One large Fitchburg hopper corresponds to four Plainfield hoppers and covers six times the area. The shaded areas in the apex of the hopper show the position which would be occupied by the contents of 1, 2 and 3, 4 Plainfield hoppers. The result is a concentration of four volumes of the best sludge on one unit area and the consequent increase of gas evolvement within this unit area. The effect of evolvement of gas from sludge concentrated on a comparatively small area, as compared with evolvement over a scattered area, is indicated in Fig. 6. To the right the minute pin-point gas bubbles are shown to be held individually by the entangling solids; at the left the four small bubbles occupying the space formerly occupied by one bubble must necessarily be nearer each other, and so will collide to form large bubbles. These bubbles, having sufficient buoyancy to rise with considerable momentum, are, with their burden of solids, diverted by the trapped slot, whereas the few individual small bubbles, weighted down with solids, hang suspended and, having no momentum, cannot be deflected from the slot. Whether this effect of concentration of sludge and gas is accompanied by lowering the bottom of the tank, by raising the slot, or by use of a few large hoppers replacing several small ones, comes to much the same thing.

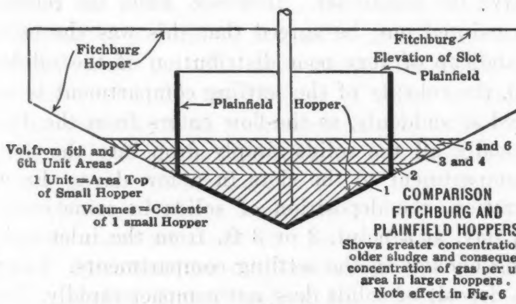


FIG. 5.

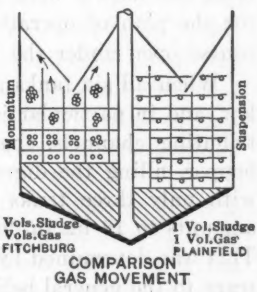


FIG. 6.

By good fortune Dr. Imhoff, the inventor of the tanks, spent an afternoon late in September, 1923, at the Plainfield plant. His reaction to the design and operation of the Plainfield tanks is of interest.

In anticipation of his visit, the superintendent had all the slopes, slots, and settling compartment surfaces cleaned with particular care 24 hours before his arrival and then gave orders that the surfaces were not to be touched again until Dr. Imhoff had seen them 24 hours later. The moment he came, he demanded, "What is that black mass doing on the settling compartment?".

It was explained that this was the very question which he was expected to answer. Samples were then pumped from every part of the digestion compartment, showing that the condition previously described existed throughout. The sludge level was well below the neutral zone.

When Dr. Imhoff saw the plans, he stated that the sludge depth below the slot was far too limited. He was very emphatic in stating that good results could never be expected with such an arrangement—that it is useless to provide a given number of cubic feet of space without regard to the depth factor. As to the value of depth below the flow line, his answer was, "That is nothing, you must have depth below the slot". It is to be noted, however, that in his writings* he has laid stress on the value of the total depth.

Another criticism which he made was that the opening below the curtain wall should have been closed. He stated:

"You cannot successfully work two tanks over one bottom; and that is what you are trying to do. However, if you raise the slot so as to get more depth below it, you may keep the opening submerged in the heavy ripe sludge, in which case it would not do much harm".

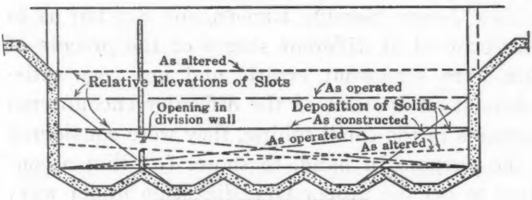
He also emphasized that the communicating opening between hoppers in the same row should be either lowered so as to be always submerged in the ripe sludge, or closed.

As to the operation—working only three tanks at one time—Dr. Imhoff pointed out that this plan was wasteful as a great and sensitive organization had been built up within the digestion chamber capable of handling a certain quantity of organic matter daily. During the period of intense feeding the organization attempts to build itself up to the work in hand and just as it is about established on this basis the supply of food is cut off and the vast organization goes hungry. He stated that it would be just as reasonable to try to feast two days a week and starve the remainder. However, when the reason for the plan of operation was pointed out, he agreed that this was the only course open under the local handicap of very poor distribution of the solids.

When all six tanks are used, the velocity of the settling compartment is so low, and in particular is reduced so suddenly, as the flow enters from the distributing channels, that practically all the solids simply drop into the first hopper, filling the digestion compartment to the slots in a few days. Even with only three tanks in operation, the deposition of solids in some cases amounts to 12 in. in depth per day, at a point, 2 or 3 ft. from the inlet end. This was determined by hanging buckets in the settling compartments. Contrary to the general belief, this deposit of solids does not compact rapidly, the first tendency toward mechanical compaction being counteracted by gas distention, as soon as disintegration sets in, so that solids deposited to such depths may not shrink appreciably for a period of 12 to 18 days. This factor, of course, is not effective where a thin deposition of fresh solids is made over a large area of ripe sludge. In the latter case the fresh solids almost at once become engulfed in the mass below so that it is, in fact, difficult to secure any sample of the freshly deposited solids from a digestion chamber which is working properly. (The latter statement was also expressed by Dr. Imhoff.)

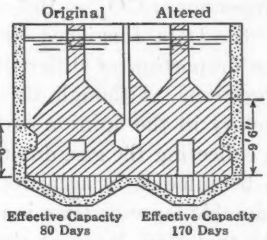
* "Eight Years of Imhoff Tank Design and Operation," *Engineering News*, Vol. 75 (1916), p. 14.

A longitudinal section of the tank is shown in Fig. 7, indicating the sloping inlet and the distribution of the solids under three conditions: (*a*) The plant as originally constructed, with six tanks operating; (*b*) only three tanks operating so as to increase the velocity by cutting down the total cross-section; and (*c*) a distribution which it is hoped to attain by raising the slots (cutting down the cross-section of the tank at the point of inlet), using submerged orifice inlets, and making possible the use of six tanks in parallel.



DISTRIBUTION OF SOLIDS
PLAINFIELD, N. J.
IMHOFF TANK

FIG. 7.



CAPACITY PLAINFIELD TANK
BEFORE AND AFTER ALTERATION
Compare Fig. 10

FIG. 8.

Similarly, the distribution across the section of the tank is subject to some measure of control. As Mr. Gavett points out plain mechanics would account for a better distribution of solids with the increase of the distance through which they have to fall after passing the slot.

In Fig. 8 is shown the cross-section of the Plainfield tanks as built and as altered to meet the criticisms of Dr. Imhoff. The inlet will be through a submerged orifice placed at the bottom of the old slope, so as to come within the restricted cross-section in the hope that the increased velocity will carry some of the solids forward to the second and third hoppers. The bottom of the bulkhead containing the orifice will be movable for the removal of solids lodged behind it. The Operating Committee of the Plainfield plant was impressed with the possibilities of relief by using these ideas and felt that in the interests of all concerned the proposition deserved a fair trial, so that it has authorized the alteration of one tank on an experimental basis, which change is now under way.

With these changes made, Dr. Imhoff expects no further difficulty in operating the plant satisfactorily; he specifically includes in his definition of satisfactory operation the elimination of the excess labor, and agrees with the speaker that an Imhoff tank which operates by virtue of excessive labor or of mechanical devices for keeping down scum is not a satisfactory installation.

Dr. Imhoff did not advance any theory in support of the changes advocated, and his recommendations were not accepted without considerable study. The available data for this study included:

1.—Operating data on the Plainfield tanks over a period of nearly seven years, showing: (*a*) the distribution, which had been very poor at first, to be somewhat improved by operating only half of the tanks at any one time, and the improved distribution of solids to result in a greatly improved digestion; (*b*) the volumes of solids deposited, calculated from cone glass readings over

these years; (*c*) a check of these volumes by means of the vertical deposition of solids in buckets hung in the settling compartments at various points*; (*d*) the quantity of ripe sludge removed in 7 years; and (*e*) the length of time required to digest solids to a 90%–95% sludge, the best sludge obtainable in these tanks (the time equals about 30 days).

2.—Published data from various sources indicating that digestion tanks in good working order may be expected to evolve gas in about the following proportions: CO_2 — 10% or less, CH_4 — 85%, N, by difference — 5% or less.

3.—Data furnished by the New Jersey Sewage Experiment Station as to the proportion of different gases evolved at different stages of the process of digestion. Although these data were somewhat erratic and not very satisfactory in the opinion of the chemist on account of the difficulty encountered in collecting the gas, due to stoppages in the small tubing, they were considered to be sufficiently accurate for the purpose in hand, to show: (*a*) that a considerable lapse of time is required to get the proper CH_4 digestion under way; (*b*) that such lapse of time is roughly commensurate with the time required for minimum satisfactory digestion of sludge (30 days); and (*c*) that the final extremes are reached only after a considerable further lapse of time (about 90 or 100 days).

Of late in running one tank continuously to ascertain the ultimate outcome of such treatment, the Chemist from the Experiment Station, Mr. Leslie Campbell, found that the disturbance in that portion of the digestion chamber above the sludge proper increased until the quantity of solids in all the liquid within the compartment reached more than 3 per cent. This condition continued until about the twenty-fifth day when there was a distinct change in conditions, temporarily. It seemed as if the process was just reaching some culminating point, but evidently the compartment was not large enough to permit the completion of this change and after a brief interval the whole mass was again thrown into a state of suspension.

Weighing all the evidence in hand it seems clear that (1) the slot of an Imhoff tank should be placed with due consideration of the progress of digestion which will have been attained at the time when the freshly deposited solids (without shrinkage), have accumulated to a depth equal to the distance between the bottom of the tank and the slot; and (2) that the horizontal dimensions of the tank should be such as to distribute the solids as evenly as possible.

In Figs. 9 and 11 are indicated graphically some of the results of these studies. The curve showing the progress of digestion of a unit volume of fresh solids (Fig. 11), starts daily on that day's deposition of solids, to end 30 days later in the shrinkage of these solids to 40% of the original volume. If the compartment is sufficiently large, so that the digestion can proceed further, it seems fair to assume that, in the course of a period of time which is not at present definitely known, the moisture content of the sludge will be changed from 90%–95% to 88%, 85%, or even 80%, resulting in a further shrinkage of volume. The broken arrow (Fig. 11) indicates the general direction of such shrinkage.

* These data were furnished by the New Jersey Sewage Experiment Station.

In Fig. 10 appears the summation of the volumes of daily accumulations of solids, each decreasing according to age, as compared with the volume of uncontracted solids deposited. This gives an interesting picture of the effect of even a small change in the position of the slot. It is the speaker's belief

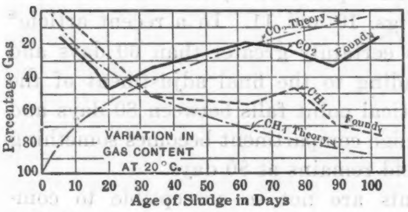


FIG. 9.

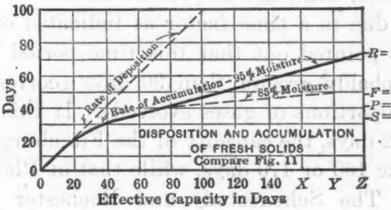


FIG. 10.

that the slot at Plainfield is just within the very critical zone where the accumulation curve begins to assume its final direction, and that the Fitchburg slot is placed just far enough above this critical zone to make the difference between success on the one hand and failure on the other.

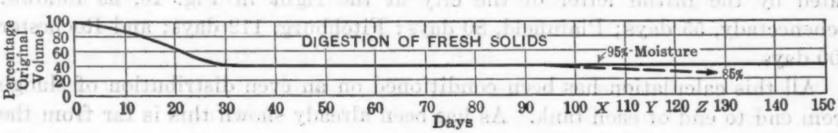


FIG. 11.

These two plants have several points in common which make them specially adaptable for comparison, as shown in Table 18.

TABLE 18.—COMPARISON OF PLANTS.

	Fitchburg.	Plainfield.
Flow in million gallons per day	3.4	3.4
Population	38 000	40 000
Sludge space in cubic feet per capita	1.47	1.22
Domestic sewage solids in grammes per day per capita	54	58*

* Allow 20 grammes for storm water at Fitchburg.

It would, therefore, seem fair to assume for purposes of comparison that they receive approximately equal volumes of domestic sewage solids per capita. At Plainfield, this amounts to 0.0305 cu. ft. per capita, or 30.5 cu. ft. per 1 000 population. The greater weight of suspended solids retained at Fitchburg would be largely made up of heavy minerals. The space occupied by such minerals is 1% of the freshly deposited solids, calculated from comparison of solids by weight and by volume.*

Assuming an equal distribution of solids in both cases, this volume would reach the bottom of the neutral zone (without any contraction) in 48 days at Fitchburg and 40 days at Plainfield. These periods are for the deposition of

* Plainfield Laboratory data.

fresh, uncontracted solids. From Fig. 10 the actual capacity for sludge of the consistency found at Plainfield is 112 days at Fitchburg and 80 days at Plainfield.

What about the difference in density of the two sludges? It is the speaker's belief, although at present he has no positive proof of it, that the difference is due to a time factor as indicated on Figs. 10 and 11. In a recent article* he pointed out that this time period was certainly greater than 60 days and probably greater than 90 days (corresponding to the final adjustment of the proportions of gases evolved). If this critical point falls between 80 days and 112 days, the capacity of the Fitchburg sludge compartment becomes something like 160 or 170 days, while that at Plainfield remains at 80 days.

The Schenectady and Rochester plants are not so susceptible to comparison; and yet the weight of suspended domestic solids is near enough in all cases to justify the application of the same unit volume of solids in all four cases to see where these two plants fit into the picture. Schenectady has 33 days' capacity for uncontracted solids and Rochester 72 days'. The position of the bottom of the neutral zone in terms of effective capacity has been indicated by the initial letter of the city at the right in Fig. 10, as follows: Schenectady, 55 days; Plainfield, 80 days; Fitchburg, 112 days; and Rochester 200 days.

All this calculation has been conditioned on an even distribution of sludge from end to end of each tank. As has been already shown this is far from the fact at Plainfield and, presumably, at Schenectady, where the communication between hoppers is much restricted. This suggests another factor in favor of the two successful tanks.

The author, calculating from the weight of suspended solids, has taken 1.83 cu. ft. per person (60 days at 0.0305 cu. ft. per capita) as the required sludge space. In calculating from the volume of fresh solids deposited, the speaker feels that 60 days' capacity of such solids should be the minimum allowance. It is true that Fitchburg is successful with an allowance of only 48 days, but this is too fine a line to draw in designing a plant. Again, Mr. Eddy computes 2.75 cu. ft. of ripe sludge per person to be removed per annum. The result of actual measurement at Plainfield is 0.1 cu. yd. per person per year, or 2.7 cu. ft. It is strange, however, that although these figures check so exactly, Mr. Eddy has calculated on a sludge containing 15% of solids and the speaker's experience has been with sludge containing barely half as much.

It is hard to explain this discrepancy. To designate the condition of a sludge in percentage of solids is inadequate. There is no escaping the mathematical correctness of this method of expression, but the idea that a sludge of 15% of solids simply contains twice as much solids per volume as a sludge of 7.5% of solids is an incomplete grasp of the situation. There is something more than a mechanical concentration. Sludges of less than 6% solid content may be considered as merely a mechanical mixture, but when through the natural process of digestion the solid content reaches 7%, there is a definite change in the character of the sludge; it might almost be stated "in the structure of the sludge", but up to this point the sludge has no struc-

* *Public Works*, Vol. 54 (1923), p. 363.

ture—it is a heterogeneous mass of solids and water. Above 7% it may be said to assume a sort of structure—there is a re-arrangement of the particles, the mass becomes viscous and this results in a capacity to hold great volumes of gas. This change is progressive through a range from 7 to 12% of solids at least. Although nothing better than the percentage statement of solids is offered at this time, such an expression is inadequate and it is to be hoped that something better may be developed.

It is reasonable to suppose that the considerable volume of gas which the heavier sludges are capable of entraining could materially effect the volume of space occupied by a given weight of solids in a given dilution of water. This matter might well be added to the list of subjects on which more light is required.

David A. Hartwell,* Esq. (by letter).†—The author has covered his subject so thoroughly that there seems little to be added; there are a few matters, however, that may with interest be further discussed. The writer has a feeling that the same type of tank construction might act differently in different localities due to varying conditions, such as the climate, the composition of the sewage and the theory of the operator as to the causes of certain objectionable conditions and the methods of correcting them.

In designing the Fitchburg tanks provision was made for treating the sewage from a population of 55 000 with a theoretical retention period of 3 hours. During 1923 the retention period varied from 10 hours in December to less than 5 hours in March, the weighted average for the year being 6¼ hours, as compared with 6½ hours in 1922 and 8½ hours in 1921. Possibly the long period of retention in the sedimentation compartment at Fitchburg may cause a settlement of certain solids that would not be effected during a shorter period and this may have considerable effect on the process of digestion.

In summer the water is cooler in a deep well than in a shallow one. Is it not reasonable to assume that the temperature of the sewage in a shallow tank would be affected more by changes in the temperature of the air than the sewage in a deep tank, being colder during the winter months and warmer during summer months? If this were a fact as well as a theory, would not digestion in a deep tank likewise be more active during the colder months and less active during the warmer months? According to Fig. 1,‡ an increase of 3° in temperature in the digestion compartment would entail a considerable increase in the activity of the digestion. Recent tests at Fitchburg made after a period of zero weather for several days indicate a temperature between 44° and 45° in the digestion compartments of all the Imhoff tanks at a point about 4 ft. below the slots. This is probably a minimum temperature under normal winter conditions.

It has been the custom at Fitchburg to draw sludge from the tanks as early as conditions on the drying beds warrant. This removes any digested sludge early in the season and leaves the digestion compartment in the best possible condition for the maximum work to be done during the warm months.

* Commr. of Public Works and City Engr., Fitchburg, Mass.

† Received by the Secretary, February 25, 1924.

‡ *Proceedings*, Am. Soc. C. E., May, 1924, p. 627.

In 1921 as much sludge was removed in April as during any other month except June and October; in 1922 the April sludge removal was only exceeded by that of July; and in 1923 that in May was greater than in any month except June and July. The removal of sludge from the tanks as early in the season as possible has been a large factor in the elimination of foaming during the last three years.

In the first years of operation of the Fitchburg plant the disposal of the secondary tank sludge was a serious problem. The design of the plant made provision for pumping this sludge into the influent pipe to the Imhoff tanks, in the expectation that it would mix with the sewage, to settle and be digested with the sewage sludge. This method delivered the secondary tank sludge into the sedimentation compartments of all the tanks while they were in normal operation, which procedure was objectionable on account of (1) the light flocculent foam that covered all the tanks for a number of days, (2) the reduced efficiency of the trickling filter and (3) the unsatisfactory final effluent. A later method was to pump the secondary tank sludge into one or two tanks instead of all five. This necessitated shutting down the whole plant and turning all the sewage into the river while the secondary tank sludge was being pumped—an objectionable method. Finally piping was installed to carry this sludge from an outlet 18 in. below the slots directly into the digestion compartment of one tank while the whole plant is in operation. The sludge can be delivered to any or all of the hoppers of this tank. This method has now been in operation for nearly three years with highly successful results. There has been absolutely no foaming in this tank, which has digested all the sludge from the secondary tanks. The mixture of secondary tank sludge with sewage sludge is more rapidly digested than sewage sludge alone and there is a great increase in the volume of sludge pumped yearly from this tank. During the last three years 43% of all the sludge pumped from the five Imhoff tanks has been pumped from this one tank.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JOHN BOGART, M. Am. Soc. C. E.*

DIED APRIL 25, 1920.

The death of John Bogart marks in the recollections of civil engineers, both young and old, the passing of one of the great engineering minds of the time. His life, from early youth to the age of 84, was devoted to the profession of engineering, and he leaves behind him a record of achievement in the inception, design, and construction of work, that has been excelled by few engineers of his generation.

John Bogart was of Dutch descent, his ancestors having come from Holland to the United States about 1641. They originally settled in Albany, N. Y., where he was born on February 8, 1836, the son of John Henry and Eliza Hermans Bogart.

He received his early education in the Albany Academy, which institution was noted for its thorough training. At that period, the two great prizes of the academic year were the Van Rensselaer Classical Medal and the Caldwell Mathematical Medal, given to the best student in each of those branches of study. Mr. Bogart was the first person awarded both these medals in one year.

From the Academy he went to Rutgers College, at New Brunswick, N. J., where many sons of Albany Dutchmen had received their collegiate education. He was graduated in 1853 with the degree of Bachelor of Arts. The College subsequently conferred on him the degree of Master of Arts and, in 1912, the degree of Doctor of Science.

Mr. Bogart began the study of law with the intention of practicing that profession, but on leaving college his health was delicate, and to secure the advantages of active exercise he entered at once the Corps of Engineers of the New York Central Railroad and was actively engaged for several years on the improvements of its lines. A large part of his duties was in connection with the construction of the direct road between Syracuse and Rochester, N. Y., through Clyde, Lyons, and Palmyra, which affected a saving of twenty miles as compared with the older line by way of Auburn, Geneva, and Canandaigua. This experience in engineering work established his choice of a profession.

His next service was as an Assistant in the Engineer Department of the State of New York, in which position he was employed on the works of reconstruction and enlargement of the canals in the Eastern Division of the State. For some time, he occupied one of the offices in the State House over which, thirty years afterward, he presided as State Engineer.

The construction of Central Park in New York, N. Y., was at that time being planned. This work involved important engineering in the roads, tun-

* Memoir prepared by Herbert Spencer, Esq., and Charles A. Pohl, M. Am. Soc. C. E.

nels, bridges, drainage, and water system, as well as the artistic element of landscape treatment which appealed especially to Mr. Bogart's cultivated taste. Here, he was engaged until the beginning of the Civil War and became deeply interested in the development of urban and suburban park improvements. He was afterward connected with many such improvements in various parts of the United States, notably as Chief Engineer of the Brooklyn Park Commission in the construction of Prospect Park.

The urgent demand of the Government at the outbreak of the great Civil War for the services of the young men brought immediate response from Mr. Bogart and from his only brother, James Henry Bogart, who served through the war to the siege of Port Hudson, where, as Major of one of the New York regiments, he was killed while leading his troops. John Bogart was invited to act as an Engineer and served throughout the war, being stationed mostly in Virginia. He had charge, under Brig.-Gen. William H. Brewerton, of the construction of the heavy fortifications on the Rip Raps, an island in Hampton Roads, which, in connection with Fort Monroe, guarded the mile-wide channel from the ocean to the James River and to Norfolk, Va. He was present at the memorable engagement between the ironclad, *Merrimac,* and the *Monitor,* then called Ericsson's Battery, witnessing from the mast of one of the ships the fight which revolutionized naval warfare. He was also engaged in military operations at a number of other points during the war and at its close remained for a time in Government service. He was soon called, however, to the active duties of his profession in civil life and resigned in 1866.

While a detailed account of the work executed or investigated by Mr. Bogart would be too lengthy, the rather marvelous range of projects submitted to him for advice may be mentioned in order to show his grasp and understanding of engineering in its larger aspects.

In connection with the development of park areas, he served as Chief Engineer or as Consulting Engineer on work of great importance in many cities. As Chief Engineer of the Department of Public Parks in New York City, from 1872 to 1877, he constructed what are probably the best known parks in the United States. His work also included parks and boulevards in Albany, N. Y., New Orleans, La., Nashville, Tenn., Baltimore, Md., Chicago, Ill., Essex County, New Jersey, and Riverside Drive, New York.

Mr. Bogart served as Deputy State Engineer and Surveyor of New York State during 1886 and until the summer of 1887, when he resigned that position. He was elected State Engineer and Surveyor in the fall of that year, assuming office on January 1, 1888.

One of his early designs was for a bridge at Hartford, Conn., considered at the time to be one of the first flat arches in America. In 1887, Mr. Bogart was engaged as Chief Engineer in charge of the construction of the beautiful Washington Bridge over the Harlem River in New York. He also served as Advisory Engineer for the original Rapid Transit Commission of New York, and, as such, was engaged in the preparation of plans and contracts for the first subway system. He also prepared plans for tunnels under the Hudson River to bring those railroads that had terminals in Jersey City and Hoboken,

N. J., into a proposed Union Station at 42d Street and Eighth Avenue, New York. The Steinway Tunnel to Queens, now operated as a part of the New York Subway System, was included in the plan for bringing the Long Island Railroad into the proposed Union Station. A bridge over Hell Gate and a Connecting Railroad to Bay Ridge, Long Island, with a ferry to Bayonne, N. J. (substantially as built later), was projected as part of this design.

The foundations of some of the earlier of the large buildings in New York also engaged his attention, such as difficult problems of deep foundations, notably those of the Commercial Cable Building, Queens Insurance Building, Hanover Bank, Savoy Hotel, New York Life Insurance Building, New York Athletic Club, and St. Agnes Chapel.

In 1890, Mr. Bogart became much interested in hydro-electric development which was then in its early stage, and was in touch with most of the projects reported on or constructed. He was appointed Consulting Engineer of the Cataract Construction Company, a subsidiary of the Niagara Falls Power Company. At that time, little was known of power transmission, but certain European developments had attracted some attention; with the view of utilizing some method for the transmission of the vast power of Niagara, he investigated such European installations as could then be found. Accompanied by Mrs. Bogart and the late Francis Lynde Stetson, Vice-President of the Niagara Falls Power Company, he traveled over a large part of Europe studying existing methods of power generation and transmission. Four methods of water-power transmission were found to be in use: (1) By means of manila or wire rope, in some instances taking power nearly a mile, which method was that in most general use; (2) by hydraulic transmission through pipes; at Geneva, Switzerland, electric lighting dynamos were operated by water piped under pressure through city streets; (3) by pneumatic transmission, which method was in extensive use in Paris, France, Birmingham, England, and elsewhere; and (4) by electricity.

In all France, they found only three examples, and, in Switzerland, only one example, of electric transmission in actual use. At Domène, opposite the Grand Chartreuse in the Dauphine Alps, the power for a paper-mill was drawn from a glacial stream in the mountains four miles away. Here was found the precedent which largely influenced the final decision as to the system to be adopted at Niagara. Mr. Bogart was actively engaged in the planning and construction of this pioneer development in America from its inception to its completion.

The tremendous field of hydro-electric development and the possibilities of electric transmission appealed very strongly to him. In 1899, he investigated the possibilities of utilizing the power of the St. Lawrence River at Massena, N. Y., and jointly with Messrs. Kincaid, Waller, and Manville, of London, England, he designed and built the plant of the St. Lawrence Power Company at that place.

As a Consulting Engineer on hydraulic developments, Mr. Bogart's reputation was now established, and brought him extensive work in various parts of the country. He served as Engineer of the plants of the Cascade Power Company in British Columbia, the Lake Superior Power Company, the Knox-

ville Power Company, the Youghiogheny Power Company, the Niagara, Lockport, and Ontario Power Company, and the Sault Ste. Marie Power Company. He also designed and built the plant of the Atlanta Power Company at Bull Sluice, Ga., and many others.

Probably Mr. Bogart's greatest contribution to hydro-electric development was as Chief Engineer of the Chattanooga and Tennessee River Power Company, on a 60 000-h. p. plant in the Tennessee River near Chattanooga, which was the first installation using private capital to improve the navigation of a river under Government control in return for the power created. This work was particularly difficult on account of the great variation in head on the turbines and the difficult foundations for the dam. It was found necessary to resort to reinforced concrete pneumatic caissons in making the excavation and afterward to incorporate these as part of the dam. As far as known, this was the first instance where pneumatic caissons had been used for this purpose.

In 1913, Mr. Bogart formed a partnership with Charles A. Pohl, M. Am. Soc. C. E., which was continued until his death on April 25, 1920. The business is still maintained by Mr. Pohl under the firm name of Bogart and Pohl.

In 1870, Mr. Bogart was married to Miss Emma Cherrington Jefferis, of West Chester, Pa., who survives him.

He served on many boards and commissions, of which the following is a partial list: Nicaragua Canal Commission, Railroad Terminal Commissions of Buffalo, N. Y., and Toronto, Ont., Canada, Croton Aqueduct Commission, New York State Board of Health, etc. He was appointed a member of the Municipal Art Commission of New York City, and was the only civilian member of the Board of Engineers appointed by the President of the United States to examine the feasibility of constructing a deep waterway from Chicago, Ill., to the Gulf of Mexico. Mr. Bogart was also delegated by the President to represent this country at the International Navigation Congresses held at Düsseldorf, Germany, in 1902; at Milan, Italy, in 1905; and at St. Petersburg, Russia, in 1908. At Philadelphia, Pa., in 1912, he served as Chairman of the Inland Section of the Congress.

He was a Lieutenant-Colonel of Engineers of the National Guard of New York State and served for ten years on the Staff of Gen. Charles P. Roe.

Mr. Bogart was employed as expert in many Court cases. His work was of such varied nature that his testimony was equally varied, and his clearness of expression, uniform courtesy, and good humor made it extremely valuable to his clients.

Mr. Bogart believed the truly great engineer should be able to harmonize and correlate all phases of engineering so as to make his work of the greatest service to mankind. The following extract from a speech delivered by him before the Engineers' Club of Rochester, N. Y., serves to illustrate his ideas on the question of specialized engineering:

"As perhaps the oldest member of the profession here to-night (though indeed you are making me feel young again by your courtesy), I may recall the difference in the larger appreciation of the work of the Engineer. It was always necessary, essential, fundamental in the accomplishment of great undertakings, but not so very long ago the men who were talked about, recognized by the great public in connection with material progress and the success

of great undertakings, were the men who managed the finances. In fact, Engineers used to be willing to permit their functions to appear to be subordinate. The Engineer had not 'found himself'. He did not quite appreciate the fact, or, at least, did not insist upon large recognition of the fact that what he gave to an enterprise, was the art, the skill, the 'know-how', without which neither money nor material, nor energy could be so put together, each in its right relation, like the various parts of a timepiece, and thus make the enterprise a working success.

"I think we may fairly say that conditions have been changed. The industrial history of the recent years of progress has been in large part made by the achievements of Engineers. You do not need a recital of instances. They stand out large, and the world is not unconscious of these facts. The world desires to know about feats of Engineering, and, gentlemen, we should not, in justice to each other, hide our brilliant light.

"The phases of Engineering have become wonderfully varied, not only in the great divisions, mechanical, mining, electrical, all sub-phases of civil engineering, but all these demanding special study of interdependent arts. I sometimes fear that some of us specialize too much for the general good. We must not forget that each specialization is only a part, and that somebody must combine all essentials into such harmonious results as should characterize good engineering, and we must insist that the one who accomplishes final results is the great engineer."

During his long and interesting career, Mr. Bogart was associated with most of the great engineers of the Nineteenth Century, and his genial disposition, combined with his remarkable fund of knowledge, drew men to him for advice on the most diversified engineering subjects. This is the more remarkable in this day of highly specialized engineers. In his career, however, Mr. Bogart seems to have covered the field of engineering work, from railroads, canals, water-works, tunnels, parks, and bridges, to great hydro-electric developments, dams, and inland waterway and irrigation projects. He was in the true sense an organizer and, as such, in great demand. His profession was a labor of love and he gave to his work the painstaking care which was marked more by the desire to serve his fellow-beings than for financial return.

The Society owes a great deal to the memory of Mr. Bogart for his untiring work in its early days. He served as a Director from 1873 to 1875, as Treasurer from 1876 to 1877 and from 1891 to 1894, and as Secretary from 1878 to 1890, and was largely responsible for shaping many of the ideas and policies which have served to give the Society its present standing.

Mr. Bogart was elected a Member of the American Society of Civil Engineers on February 17, 1869.

WILLIAM DEXTER BULLOCK, M. Am. Soc. C. E.*

Died April 30, 1923.

William Dexter Bullock was born in Rehoboth, Mass., on April 17, 1850, the son of William R. and Hannah G. (Carpenter) Bullock, both of whom

* Memoir prepared by Herbert E. Sherman and Elmer W. Ross, Members, Am. Soc. C. E., and Robert L. Bowen, Assoc. M. Am. Soc. C. E.

were of New England descent. Mr. Bullock was a graduate of Union College, Schenectady, N. Y., and received the degree of Civil Engineer in 1871.

Taking up office and field work with the Delaware, Lackawanna, and Western Railroad Company for a season, he was afterward employed in the office of the late James B. Francis, Past-President, Am. Soc. C. E., of Lowell, Mass., and in the City Engineer's Office in Lowell. In 1872, he assisted in the surveys in Washington Territory for the Northern Pacific Railroad, and, in February, 1873, he entered the City Engineer's Office in Providence, R. I., with which he was destined to be connected for the remainder of his life. For some years, he was engaged in a general way with the various branches of municipal work, but having long cherished an ambition to be a bridge engineer, he was much pleased when, in 1878, he was given charge, as Engineer of Bridges and Harbor, of the bridge and harbor work of the city by the City Engineer, the late Samuel M. Gray, M. Am. Soc. C. E. This position Mr. Bullock retained until his death.

Owing to the location of the City of Providence, the Bridge and Harbor Department is one of the important Divisions of the City Engineer's Office. The Woonasquatucket and the Moshassuck Rivers flow through the city, meeting at the center to form the Providence River. This river, together with the Seekonk River which bounds the city on the east, forms the head-waters of Narragansett Bay. The Bridge and Harbor Department not only performs the engineering required in connection with the construction and maintenance of the bridges over these streams, navigation in the Providence River, and the various City docks, but also maintains the operating forces for the draw-bridges and a construction force which does all the maintenance work, except the major contract work of reconstruction. In the forty-five years during which Mr. Bullock had charge of the Department many of the old bridges were entirely renewed or reconstructed and new ones were built, so that, at the time of his death, the Department was responsible for three swing draw-bridges and about forty fixed bridges which embraced examples of almost every type of modern bridge, including three substantial granite arches designed to harmonize with the surroundings at Roger Williams Park.

These structures show many examples of Mr. Bullock's ingenuity, especially two of the draw-bridges which are equipped with automatic safety gates of his design. By means of these gates, travel on the bridge may be controlled from the operating room on the draw-span.

About 1890, the plans for the present railroad facilities in the center of the city were adopted. The Bridge and Harbor Department supervised a part of this work which involved the construction of long granite retaining walls for the Woonasquatucket River, the removal of the mud and silt from the "Cove Basin"—a circular tide-flowed basin, 30 acres in extent—and the filling in of the basin with gravel to the elevation of the surrounding streets.

All the City docks were constructed under Mr. Bullock's direction. One built for the Highway, Sewer, and Water Departments was of wooden pile construction. The "Municipal Wharf" so-called, which was started in 1911 and completed in 1915, involved the construction on the harbor line of a granite

quay wall, 3 000 ft. long and 41 ft. high. Back of the wall a large area was filled and graded, and railroad tracks, sidings, etc., were installed.

Between 1878 and 1923, the purchasing power of the dollar varied so widely that a statement of the cost of the work done under Mr. Bullock's supervision during that period does not adequately convey an idea of its amount. It may be of interest, however, to note that during that time about $2 200 000 was spent on bridges, $500 000 on dredging, and $1 000 000 on City docks.

In addition to his work for the City of Providence, Mr. Bullock also served as Chief Engineer for the State Harbor Improvement Commission of Rhode Island during its existence from 1911 to 1918. This Commission was appointed by the General Assembly to provide additional facilities for commerce by water in Providence and Pawtucket, R. I. In Providence, it constructed a large, thoroughly modern pier, with a steel pier shed, equipped with railroad connections, immigration facilities, and cargo-handling apparatus. In Pawtucket, the Commission constructed a granite quay wall, 700 ft. long, graded the area in back, and built a large steel freight shed. This work involved the expenditure of about $1 000 000.

On October 17, 1879, Mr. Bullock was married to Miss Annie A. Taft, of Pawtucket, who died in Providence on October 31, 1899. On February 26, 1902, he was married to Miss Florence S. Clapp, of Providence, who, with a son and a daughter, survives him.

He was a man of high principles and positive convictions, of marked ability and judgment, an agreeable associate, and a loyal friend. In 1886, he represented the City of Pawtucket in the Rhode Island House of Representatives. He attended the Central Congregational Church in Providence.

Mr. Bullock was elected a Junior of the American Society of Civil Engineers on September 5, 1887, and a Member on July 4, 1888.

CLINTON SUMNER BURNS, M. Am. Soc. C. E.*

Died April 1, 1924.

Clinton Sumner Burns was born at Waverly, Iowa, on October 26, 1871. He was the son of Herman H. and Laura Root Burns.

His early education was obtained in the public schools at Port Byron, N. Y., where he lived with his grandmother, his mother having died when he was nine years of age. He was graduated from the Port Byron Academy at the age of twenty and succeeded in winning a scholarship at Cornell University which he entered as an engineering student. While attending Cornell, he acted during vacations as Assistant City Engineer of Niagara Falls, N. Y.

After two years at Cornell, Mr. Burns went to California to enter Leland Stanford University. Before leaving Cornell, he had taken a Civil Service examination for the position of State Leveler for New York State and after arriving in California was notified of his appointment, his being the highest grade among a number of applicants. He returned to New York to accept this

* Memoir prepared by Robert E. McDonnell, M. Am. Soc. C. E.

position and was engaged on work on the Erie Canal and on other State canals. At the end of the year, he returned to Stanford University from which he was graduated in 1897 with the degree of A. B. in Engineering.

Mr. Burns secured his education and technical training at considerable personal effort, having sought employment at the various schools he attended as a means of defraying his expenses. While at Stanford University, he held the position of Assistant City Engineer of Palo Alto, and, on Saturdays and at odd times, established grades and made lot surveys. During his vacations, also, he secured engineering engagements, thus paying his way through the University.

After his graduation, Mr. Burns went to Kansas City, Mo., where he entered the employ of the City Park Department, under Mr. George Kessler. He held this position until April 1, 1898, when he and Robert E. McDonnell, M. Am. Soc. C. E., a classmate at Stanford University, formed a partnership under the firm name of Burns and McDonnell and began the private practice of municipal engineering, specializing in water supply and sewerage.

In 1919, the name of the firm was changed to Burns and McDonnell Engineering Company, four of the older employees having been admitted to membership in the organization. In 1923, a branch office was inaugurated at Los Angeles, Calif., to carry on the same class of work. Mr. Burns died on April 1, 1924, on the twenty-sixth anniversary of the establishment of the firm.

He gave his personal attention to the appraisal of a large number of waterworks plants that changed from private to municipal ownership, the more important being those at Council Bluffs and Des Moines, Iowa, Everett, Wash., Billings, Mont., and Riverside, Calif. He lived at Riverside for about a year, personally planning and directing the rehabilitation of that water-works plant.

Mr. Burns was of a mechanical turn of mind, given to research and investigations of various kinds, and he often maintained testing apparatus and equipment at his home for the special study and analysis of engineering problems. Several patents were granted to him for various devices, one a joint for sewer pipe, another for some special railroad equipment, etc. These patents were sought and secured not with a view of financial profit, but more for the love of and satisfaction gained in the work of investigations.

His clear thinking, love of mathematical problems, and analytical treatment of all engineering questions, was a matter of much comment on the part of his associates and all those who knew him personally. His profession called him to all parts of the country and involved work in practically every State in the Union.

Mr. Burns was a frequent contributor to the engineering press and had written a number of papers on valuation questions, of which that entitled, "The Valuation of Sewerage Systems", is one of the few—perhaps the only published article—covering this particular class of utilities. The paper relates to personal experiences during the appraisal of a number of sanitary sewerage systems throughout the Southern States, where such utilities are frequently owned by private companies. He was also the author of a pamphlet entitled, "The Relationship between Finance and Depreciation", and another pamphlet descriptive of the Sultan Water Supply of Everett, Wash., a gravity water supply project

diverting the Sultan River through several tunnels and under river crossings in a 28-mile pipe into the City of Everett.

In 1905, nearly twenty years ago, Mr. Burns wrote an interesting article entitled, "The Diary of a Young Engineer", in which he recorded anecdotes and the early experiences of a young engineer struggling past the first few mile-posts of his professional career. These experiences show that at an early age he had a high conception of the ethics of his profession and his article closes with the comment: "The practicing engineer cannot hope to secure much business along the branch of the consulting profession until he has acquired a reputation of absolute honesty, untainted by commissions or connections with construction companies." He further states in this article that the young engineer may give his terms and if he is true to his profession his price will be high enough to enable him to do the work thoroughly. This interest in professional fees continued throughout his life, for he was Chairman of the National Committee of the American Association of Engineers on Services and Fees for Practicing Engineers.

At the time of his death, Mr. Burns was serving Kansas City as one of the five members of the Board of Review on the plans for the $11 000 000 water plant for the City, which important improvement he had been actively urging for many years.

He became at an early age a member of various engineering organizations, having been one of the founders of what is now The Engineers' Club of Kansas City. He was elected a member of the New England Water Works Association in 1909 and, for several years, had been a member of the American Institute of Consulting Engineers. He was also an Honorary Member of the Sigma Xi, having been elected in 1905 from the Stanford University Chapter.

Locally, Mr. Burns was a member of several civic organizations. His chief recreation was golf, and he was a member of the Milburn Golf and Country Club and the Community Golf Club. His daughter, Miriam, is City Champion of Kansas City, State Champion of the State of Missouri, and, in 1923, won the championship in the Women's Western Golf Tournament, and Mr. Burns took keen delight in following her contests. Besides his daughter, his widow, Mabel M. Burns, survives him.

Mr. Burns was elected an Associate Member of the American Society of Civil Engineers on February 1, 1899, and a Member on January 3, 1905.

JUSTUS VINTON DART, M. Am. Soc. C. E.*

DIED SEPTEMBER 26, 1923.

Justus Vinton Dart, the son of Eben E. and Juliet (Hurlbut) Dart, was born in New London, Conn., on October 11, 1849. His ancestral line probably leads back to Richard Dart, who is recorded as having been a resident of New London in 1664 and the head of a large family.

* Memoir prepared by Herbert E. Sherman, M. Am. Soc. C. E.

Mr. Dart's early education was begun in the public schools of his native city and completed at the Department of Yale University now known as the Sheffield Scientific School, from which he was graduated in the Class of 1871.

In May, 1872, he entered the office of Charles E. Paine, City Engineer of Providence, R. I., and for some years was engaged in the various lines of municipal work. Much of his time, however, was devoted to surveys, plans, and estimates for the construction of new streets required by the rapidly growing city; in 1877, the newly elected City Engineer, the late Samuel M. Gray, M. Am. Soc. C. E., in re-organizing the office, placed Mr. Dart in charge of that work. Later, in 1881, he became the head of the Highway Department of the City Engineer's Office, his duties including the design and adjustment of street grades, the preparations of plans, estimates, and specifications for street building and paving, the supervision of highway work in progress, and the location of street railways.

During Mr. Dart's term of service for the city, important changes were taking place. In area, the municipality gained approximately 100%; in population, the increase was, perhaps, two and one-half times greater. Generally speaking, highway travel and traffic were tremendously stimulated by the invention of new methods of locomotion and the urgent need for smoothly surfaced streets with ample stability for the transit of heavier loads inspired a corresponding activity in the invention of new methods and the use of new materials for highway construction.

In the early Seventies, the more important streets of Providence were paved with cobblestones laid on a sand foundation, while those subject to lighter traffic were surfaced with screened gravel. The use of the steam roller for consolidating the foundation courses and finishing the wearing surfaces had only recently begun. During this decade, the matter of improved street surfaces received much attention, and the use of granite blocks laid on a sand or gravel foundation came into vogue. Some rather unsuccessful experiments with wooden blocks and briquettes of crushed limestone and asphalt were also undertaken. A careful study was made in preparing for the improvement of Westminster Street—an important thoroughfare in the business section—and, in 1880, the work was performed under the supervision of Mr. Dart. It was later stated by him to have been the "first permanent pavement constructed by the City of Providence".

The specification called for a 5-in. foundation course of natural cement concrete, on which a bed of clean, dry gravel, $1\frac{1}{2}$ in. thick, was evenly spread, to receive granite blocks, laid with open joints. The joints were then filled with clean, hard, dry sand—artificially dried if necessary—and the blocks rammed to an even surface, after which the joints were saturated with a heated mixture of coal-tar pitch and creosote oil.

This pavement stood up under heavy traffic for more than 40 years, being replaced by sheet asphalt in 1922, when the underlying foundation was found to be intact and to require only the additional material for bringing the thinner asphalt surfacing to the established grade.

From this time, the use of improved methods, gradually introduced, was steadily followed. At the close of Mr. Dart's connection with the Highway

Department, the City had about 260 miles of municipal streets of which, about 70 miles were paved in accordance with modern practice and less than 3 miles were of cobblestone construction. Of newly accepted streets, 8 miles remained unbuilt, and the residue, located chiefly in the outer districts, were of water-bound or bituminous macadam.

In the spring of 1915, Mr. Dart was appointed Superintendent of Highways, assuming the charge and daily inspection of the outside street construction, in addition to the direction of his Department in the office of the City Engineer. He resigned from the employ of the city on July 1, 1916, and resided in Thompson, Conn., until his death.

In a review of Mr. Dart's professional career, one may consider him a type of the great army of municipal engineers, who steadily labor, year after year, in the interest of the public. Earnest, honest, industrious, intelligent, and capable, their virtues are not always appreciated. The merest details of their minor duties call for painstaking care and accuracy, while trained judgment and scrupulous impartiality are no less required in matters of greater importance.

Skilled in his vocation, diligent in the prosecution of business, Mr. Dart, was courteous and agreeable to his associates, fair and considerate to his subordinates, and generous and loyal to his many friends.

He was married on January 1, 1894, in New London, to Miss Alice F. Howard, who survives him.

Mr. Dart was elected a Member of the American Society of Civil Engineers on April 4, 1900.

WALTER JOSEPH FRANCIS, M. Am. Soc. C. E.*

Died March 6, 1924.

Walter Joseph Francis, the son of Joseph and Elizabeth Francis, was born in Toronto, Ont., Canada, on January 28, 1872. He received his education in the public schools of Ontario, the Toronto Collegiate Institute, and the University of Toronto, having been an Honor Graduate of the latter in 1893, with the University degree of Civil Engineer.

After his graduation, Mr. Francis became Assistant to the Chief Engineer in charge of the design of the Toronto Union Station. In 1896, he was appointed Chief Draftsman on Bridge Construction for the Central Bridge Engineering Company, but, at the end of a year's service, he entered the Department of Railways and Canals for Canada, on the works of the Trent Canal, in special charge of the design and construction of the two hydraulic lift locks at Peterborough and Kirkfield, Ont., respectively. For a paper on the subject of these lift locks read by him before the Engineering Institute of Canada, he was awarded the Gzowski Medal in 1906. At this time, he had charge of the 32 000-h. p. electric plant on the Kootenay River, at Bonnington Falls, B. C.

* Memoir prepared by J. M. R. Fairbairn, M. Am. Soc. C. E.

In 1907, Mr. Francis became Assistant Manager and Chief Engineer of the Dominion Engineering and Construction Company, at Montreal, Que., as well as Engineer to the Royal Commission on the Quebec Bridge Disaster. In 1908, he was appointed exclusive writer for *The Engineer,* published in London, England. In 1909, he reported on various hydro-electric power propositions, including works at Campbellford, Ont., and at Edmonton, Alberta, on the North Saskatchewan River. In 1910, he reported on the public utilities of the City of Edmonton; the Herald Building disaster and the Boxer Building collapse, at Montreal; the construction of the Don Siphon for main sewers for the City of Toronto; and the hydro-electric and steam plants for the City of Quebec, Que. He also represented the Canadian Society of Civil Engineers on the Committee appointed to revise the By-Laws of the City of Montreal.

In the same year (1910), Mr. Francis formed a partnership with Mr. Frederick B. Brown, which continued to the time of his death. This firm has had an active career, its members having reported on and designed numerous hydro-electric and steam plants, water supplies, sewage, and other municipal undertakings in some of the large Canadian cities. The firm was a member of the Water Board of Montreal, having been appointed at the time of its organization, and continued as such until charge of the work was taken over by the City Staff. In 1922, the firm was appointed as Consulting Engineer to the Royal Commission and charged by the Government of Ontario to investigate and make a complete report on the activities of the Hydro-Electric Power Commission of Ontario.

At the time of his death, Mr. Francis was President of the Engineering Institute of Canada, in which he had occupied a prominent position for many years, having been a most active and untiring worker in its cause.

He was Vice-President of the Corporation of Professional Engineers of Quebec in 1920; President of the Engineering Alumni Association of the University of Toronto; held membership in the Engineering Institute of Canada, the Institution of Civil Engineers of Great Britain, American Institute of Consulting Engineers (Charter Member), the Engineering Society of Toronto University (Life Member), American Public Health Association, Montreal Board of Trade, the University Club of Montreal, the National Club of Toronto, the Royal Societies Club, of London, England, and numerous other organizations. He held an appointment in the Rotary International for seven years, and was a Past-President of the Rotary Club of Montreal, as well as a member of several International Committees. He was also active in the Boy Scout and Wolf Cub movements, occupying the office of President of the Montreal Boy Scouts at the time of his death. He found time, in addition, for hospital work in two different institutions, as well as serving as Secretary of the Shriners' Hospital for Crippled Children. He was a Thirty-second Degree Free Mason, a member of the Royal Order of Scotland, and a Past Grand First Principal of the Grand Chapter of Royal Arch Masons of Quebec.

On November 26, 1896, Mr. Francis was married to Miss Laura Elizabeth Grainger, of Toronto. He is survived by Mrs. Francis and two sons, Edward and Francis.

He was one of the outstanding engineers in the Dominion of Canada, and took an active and prominent part in every phase of the development of the Engineering Profession in the Dominion of recent years.

Mr. Francis was elected an Associate Member of the American Society of Civil Engineers on May 1, 1901, and a Member on April 5, 1904.

GEORGE SEARS GREENE, Jr., M. Am. Soc. C. E.*

Died December 23, 1922.

George Sears Greene, Jr., was born in Lexington, Ky., on November 26, 1837. His father, George Sears Greene, Past-President, Am. Soc. C. E., was not only a prominent soldier in the Civil War, but also an eminent engineer, who practiced considerably in the City of New York. From him, George Sears Greene, Jr., inherited his engineering aptitude, as well as his patriotic and soldierly characteristics. He possessed the personal qualities of a gentleman, both in his professional work and in his daily life, and exemplified markedly the fine and courtly ways of the professional man who never forgot himself under any circumstances.

Mr. Greene once remarked that his New England family connections made it imperative that he should attend Harvard University for his education. After successfully completing the Freshman year, he withdrew in 1857 to take up his engineering training in practice with his father. From that time until the end of his life, he was continuously engaged in the actual conduct of engineering work of magnitude.

Mr. Greene acquired his engineering training during the infancy of the profession. His earlier engagements were on the Croton Aqueduct of New York City and in railroad work in Cuba, as well as in copper mines in the Lake Superior District. The shifting head for engineers' transits, now universally used, was invented by Mr. Greene while he was engaged, from 1872 to 1876, on an extremely accurate survey of what is now the Borough of the Bronx. This invention was made to overcome difficulties in work over rough ground requiring both speed and accuracy.

The particular work which gave Mr. Greene his eminence was in practically creating the water-front of Manhattan Island. From 1875 to 1897, he was Chief Engineer of the Department of Docks of New York City. At the beginning of his term of office, there was no systematic method of constructing the bulkhead wall. The small range of ordinary tides—about 5 ft.—made it practicable to provide effective accommodations for shipping without closed docks or any other complicated work. He therefore retained the pier system, piers projecting out into the river with slips between. Along the marginal streets, he constructed the permanent bulkhead wall.

In order to design this wall, Mr. Greene made a thorough study of the river bottom to determine the depth of the underlying rock, which he found to be at varying depths. Particularly in the North River certain parts required

* Memoir compiled from information on file at the Headquarters of the Society.

unyielding foundations at points where the depths of water and mud were great. A few bulkhead walls had been built, but they were found to be temporary, inadequate, and unsatisfactory. Mr. Greene made a thorough examination by the most effective means at his command, including the use of divers, and found the bulkhead walls in such condition that their early reconstruction was imperative. In order that the plans which he proposed might receive suitable criticism and that he himself might get the benefit of the best professional advice, he requested authority from the Dock Commission to secure "the united wisdom and experience of disinterested engineers, to examine the condition of that portion of the wall referred to in this report, and advise what remedy if any should be applied to it." This action in discharging his duty in the most effective manner by calling to his aid such professionally skilled and experienced men as the late General John Newton, U. S. A., Hon M. Am. Soc. C. E., General Q. A. Gillmore, U. S. A., M. Am. Soc. C. E., and William E. Worthen, Past-President and Hon. M. Am. Soc. C. E., was characteristic of the man.

The formal judgments of these prominent consultants combined with the results of Mr. Greene's own experience developed into the plans for the bulkhead wall which, it may be said without exaggeration, placed him, in professional prominence, practically at the head of American engineers in charge of dock and pier construction. The efficiency and permanence of this work are conclusively evidenced by the fact that the bulkhead walls which he built in those early years are still performing their functions with a minimum of maintenance and repairs. The combination of pre-cast concrete blocks with both vertical and brace piles was a conception of great merit for his purpose, and the test of many years of service has justified his confidence in that design. Those plans, supported by many years of successful experience, have demonstrated the soundness of Mr. Greene's professional judgment and the scrupulous care which he gave to the administration of the constructive work of his office.

The more credit may be given for the successful issue of such operations in those early days for the reason that Portland cement was then a foreign product, the only domestic cements available for subaqueous operations being such natural cements as the Rosendale brands. Furthermore, even where Portland cement was imported, there had been only a limited experience in determining the best composition and methods for its use in concrete work under water.

Subaqueous concrete construction on such a scale as that of the Department of Docks of New York City gave well defined and effective stimulus to this type of construction. Indeed, it is difficult with the present practically universal use of domestic Portland cements to realize the disadvantages under which pioneers like Mr. Greene had to work. In spite of this handicap, he developed a plan of bulkhead construction which is still used successfully.

The retention of the position of Chief Engineer of the Department of Docks for twenty-two years under changing administrations and other vicissitudes of office would have been impossible for one less qualified professionally, or less resolute in discharging the duties of the office to the best of his ability. It is

superfluous to state that he had his official trials. Nevertheless, he stood the shocks unshaken and in no case did the consultants called to examine or review his work ever give any material criticism. On the other hand, he was many times professionally complimented both by the officials of the Department of Docks and by prominent consultants.

Earlier City authorities had committed a great folly in filling in a large area on the North River between Perry and West 22d Streets, thus practically destroying, for commercial purposes, that part known as the Gansevoort and Chelsea Sections. These sections now contain the great piers that provide docking facilities for the largest ships entering the Port of New York. Mr. Greene was probably the strongest engineering critic of this so-called improvement of the water-front of Manhattan Island. He urged the Dock Commission to excavate the area and construct piers, which work, after much opposition, was undertaken and subsequently completed.

In 1895, the Commissioners of the Department of Docks appointed a Board of Consulting Engineers* with the request that the members thereof "review the improvements of the water front, and the plans therefor" and "submit with their report, such suggestions as they may deem proper for the betterment of such plans, with the view of securing a larger usefulness of the water front of the City of New York." This Board made several reports, the first of which was dated February 6, 1896. In this first report, the Consulting Engineers made the following observation:

"As the bulkhead wall is a retaining wall, which has to resist the horizontal thrust of the earth-filling behind it, and as both that earth-filling and the wall itself are floated in mud, the problem of providing a permanent construction becomes a very unusual one. To build a retaining wall on solid foundation is a simple problem. To build a wall to carry vertical weight only, on a soft foundation, is more difficult. To float a wall in mud, when that wall must also take a horizontal thrust, is a problem which can only be solved by care and experience, no formulas or mathematical rules being available. The wall, as now built, is a satisfactory solution of this problem. Your Board believes it to be a unique construction, one which is worthy of the most careful study, and which deserves the strongest commendation."

This statement of unqualified commendation from members of Mr. Greene's own profession was in substance a simple statement of some of the facts connected with the administration of his duties and must have been all the more gratifying to one who had spent the best years of his life in developing a great public work for the City of New York.

On December 27, 1897, the Hon. Henry F. Dimock, Dock Commissioner, made an interesting address at the "placing of a stone to commemorate the work of dock improvement between Charles and West 23d Streets, North River", in which he stated:

"This City owes a peculiar debt of gratitude to one man whose merit is only equalled by his modesty. That man is Mr. George S. Greene, Jr., the accomplished Engineer in Chief of the Department. This plan is his child. He originated it; and he alone. By his forceful advocacy he has kept it alive during all these years and has won for it the approval of all the many Com-

* This Board of Consulting Engineers consisted of Gen. Thomas Lincoln Casey, U. S. A., the late George S. Morison, Past-President, Am. Soc. C. E., and William H. Burr, M. Am. Soc. C. E.

missioners that have come and gone, and all intelligent men who have given him a hearing. This is indeed his day of triumph. He has occupied his present position for more than twenty-two years, under commissioners of every shade of politics; always giving service of the utmost professional value. A man of the highest character, of an integrity so known of all men that no breath of suspicion has ever reached him. He has stood there as an object lesson to prove that sometimes the highest professional and personal qualifications are recognized, even in official life. His retention for such a period has done infinite honor to the long line of commissioners under whom he has served. It is certainly the hope of all who are interested in the water front that the time may be far distant when the City must be subjected to the loss of his services."

This commendation from a prominent member of the Department of Docks is a marked tribute to Mr. Greene's professional character and efficiency in the discharge of his duties.

Although Mr. Greene's connection with the Department of Docks did not extend beyond the year 1897, the effective plans which he developed for the bulkhead wall of the Harbor of New York have continued in use to the great advantage of the marine commerce seeking the port.

After his withdrawal in 1897 from the position of Engineer in Chief of the Department of Docks, Mr. Greene continued in the active practice of his profession as a Consulting Engineer. His success in the water-front constructions of New York City caused his services to be sought by those public interests requiring similar works.

Probably the most prominent of those services was performed as a member of the Advisory Board of Engineers,* appointed by the Governor of New York State, for the construction of the Barge Canal. The late John A. Bensel, Past-President, Am. Soc. C. E., State Engineer during the life of this Board, sought the constant co-operation of its members, in which Mr. Greene's long experience with large public work was of unusual value. It was during this period that the Scotia Dam, the Hinckley Dam, the heavy work at Medina, the dams on the Oswego River, and other large features of the Barge Canal work, were built.

Mr. Greene was an Honorary Member of the American Institute of Architects. His club memberships also included both the Harvard Club and the Century Association, and in the latter his fine personal qualities made him a perfect centurion. His genial companionship was a special attraction in that galaxy of interesting personalities of whom he was a popular associate for many years.

He was a genuine lover of Nature and a true sportsman, and thoroughly enjoyed fishing on a preserve in the mountains of Pennsylvania and shooting on the waters of Long Island Sound.

He was a life-long member of the Protestant Episcopal Church. His religious faith was true and unvarying and was marked by the same consistency and steadfastness that characterized his whole life.

* This Advisory Board discharged its duties during nearly four years, 1911 to 1914, immediately prior to the completion of the work, and in addition to Mr. Greene, consisted of Messrs. Mortimer G. Barnes, Joseph Ripley, T. Kennard Thomson, and William H. Burr, Members, Am. Soc. C. E.

Mr. Greene was elected a Member of the American Society of Civil Engineers on December 4, 1867, the year of its re-organization. He served as a Director from 1882 to 1884, and also in 1886; as Vice-President in 1885; and as Treasurer from 1888 to 1890. During all these years he served the Society disinterestedly and faithfully and maintained his interest in its development until his death. Not many years ago he called attention with special pride to the fact that three generations of his immediate family had served as members of its Board of Direction.

JOHN WILSON HAMILTON, M. Am. Soc. C. E.*

Died April 16, 1924.

John Wilson Hamilton was born in Wishaw, Scotland, on April 16, 1870, and came to this country with his father when he was thirteen years of age. He received his technical education at Cooper Institute, New York City, having been graduated in the Class of 1898, with the degree of Bachelor of Science, receiving at the same time the Cooper Medal. In May, 1890, he received the degree of Civil Engineer.

His first position which he held from July, 1886, to May, 1892, was with F. W. Davis and Brother, of Brooklyn, N. Y. He was engaged principally in general foundry work, including the furnishing and erection of iron work for buildings and, also, in 1891, on the construction of the Company's new plant.

From 1892 to 1893, Mr. Hamilton had charge of the Carrère and Haas Iron Works, in Brooklyn, and under his supervision the steel work for a number of buildings which included the Puck Building in New York City, was manufactured. He was employed by the Empire Iron Works from 1893 until 1894, in charge of estimating and shop work. This Company was engaged in furnishing and erecting steel work for buildings.

In May, 1894, Mr. Hamilton became President of the Sherman Iron Works, of Brooklyn, which Company furnished and erected steel work for buildings, including a number of fireproof residences. From 1895 to 1896, he was employed by J. R. F. Keely and Company, General Contractors, on the construction of the German Herold Building in New York City and also on work on the Croton Aqueduct.

In 1896, Mr. Hamilton accepted a position as Designer and Estimator in charge of work, with the Hay Foundry and Iron Works, but resigned in February, 1897, to become associated with Milliken Brothers, Incorporated. While with this firm, his work was divided into three periods: From 1897 to 1899, he had responsible charge of the detailing of all kinds of steel structures, from sugar-mills to office buildings; from 1899 to 1901, he made estimations for and designed bridges and buildings; and from 1901 until 1908, he held the position of Contracting Manager for the Company.

* Memoir compiled from information furnished by Hamilton and Chambers Company and on file at Society Headquarters.

After the failure of Milliken Brothers, Mr. Hamilton organized the Hamilton and Chambers Company, of which he was President. The work of the firm consisted principally of engineering, manufacturing, and contracting. He was connected with this organization until his death on April 16, 1924, after an operation for goitre. He is survived by his widow, Emma W. Hamilton, a son, Robert B. Hamilton, and a daughter, Elizabeth Hamilton.

Mr. Hamilton was a man of many lovable qualities which endeared him to every one with whom he came in contact, in business as well as socially. He was a Shriner and a member of many clubs, including the Engineers Club, New York City, Whitehall Club, and several golf clubs.

Mr. Hamilton was elected a Member of the American Society of Civil Engineers on July 10, 1907.

EDMUND HAYES, M. Am. Soc. C. E.*

Died October 19, 1923.

Edmund Hayes, the son of Gustavus Hayes and Sarah C. Shaw Hayes, was born in Farmington, Me., on May 15, 1849. He attended the preparatory schools of his native State, taught school for a time, and entered the Scientific Department of Dartmouth College where he remained for two years. He then entered the Civil Engineering Department of the Massachusetts Institute of Technology, from which he was graduated in the Class of 1873.

Shortly after leaving college, Mr. Hayes was employed with the old Erie Railroad, Western Division, on general engineering work and barometric observations with a view to reducing grades; later, he was employed by the Passaic Bridge Company of Passaic, N. J. In 1875, he became connected with the firm of Morrison and Field, Engineers and Bridge Contractors.

When the Central Bridge Company was organized in 1878, Mr. Hayes became a co-partner with George S. Field, M. Am. Soc. C. E. Later, this Company was merged with the Union Bridge Company. These companies had a most successful career and between 1878 and 1891 built many important bridges in all parts of the world, among them the Michigan Central Railroad Cantilever Bridge over the Niagara River at Niagara Falls, which, at the time, was one of the longest steel spans.

In 1891, the Union Bridge Company was absorbed by the United States Steel Corporation and now forms an integral part of the present American Bridge Company. On the sale of the Union Bridge Company, Gen. Hayes became interested in the electrical development of water power at Niagara Falls and with other Buffalo capitalists built the third power plant on the Canadian side, that of the Ontario Power Company, which was sold later to the Canadian Government. With the same group of capitalists he organized the Niagara, Lockport and Ontario Power Company which distributes power from Buffalo, N. Y., throughout the western and central portions of the State.

* Memoir prepared by Emile Low, M. Am. Soc. C. E.

In 1914, Gen. Hayes retired from active business life, and previous to his death, he lived quietly in Buffalo, when not traveling in the United States, Europe, Egypt, and Palestine. He listed traveling and fishing as his chief recreations.

He was a member of the Buffalo Club, and of the Country Club, and a Director in the Society of Fine Arts and other institutions, such as the Marine Trust Company, and also a Trustee of the University of Buffalo.

Gen. Hayes was active in the affairs of St. Paul's Protestant Episcopal Church, of Buffalo, having served as Senior Vestryman. He was well known for his works of charity, both public and private, and gave generously to local charities. He was also a contributor to Dartmouth College and the Massachusetts Institute of Technology.

He acquired his honorary military title of General while serving as Chief of Engineers of the military forces of the State of New York on the Staff of Governor Alonzo B. Cornell. In 1913, Dartmouth College conferred on him the degree of Master of Science.

Edmund Hayes loomed large in the public life of Buffalo, although he had never held public office, nor sought it. He was one of that rare class of first citizens whose reputations filter through the community from those who know them intimately. In this manner, the people of Buffalo had gained a high opinion of Gen. Hayes' ability, liberality, and public spirit. He had been engaged in large affairs all his life and through his own enterprise had done much to upbuild the city. He was particularly valued as a councillor and his advice was much sought. His death was a distinct loss to the community.

On April 30, 1878, he was married to Miss Mary H. Warren, of Buffalo, by whom he was survived.

Gen. Hayes was elected a Junior of the American Society of Civil Engineers on April 3, 1878, and a Member on March 5, 1884.

HOWARD MURFREE JONES, M. Am. Soc. C. E.*

DIED MARCH 31, 1924.

Howard Murfree Jones, son of William Rucker and Susan Catherine (Johnson) Jones, was born at Murfreesboro, Tenn., on October 9, 1874. He acquired his technical education at Vanderbilt University, Nashville, Tenn., and at Union College, Schenectady, N. Y., having been graduated from the latter institution in 1895 with the degree of Bachelor of Science in Civil Engineering.

Early in his college life, Mr. Jones' natural industry and thrift were exhibited by the devotion of much of his leisure to remunerative occupation, without neglecting his studies. For several years, he was both student of engineering and private secretary to the Dean of the Engineering Department. He thus defrayed a considerable part of the cost of his education, working apparently under a handicap, but gaining in reality a valuable training in resourcefulness and self-control, not commonly prescribed in the college curri-

* Memoir prepared by John Y. Bayliss, M. Am. Soc. C. E.

culum. These sterling qualities, so early displayed, were retained in later years and contributed much to the high character, independence, and sense of responsibility, that dominated his whole professional life.

Immediately after graduation from Union College, Mr. Jones began his career as a Civil Engineer in the employ of the Nashville, Chattanooga & St. Louis Railway, in Nashville. He continued in this service for seven years, rising to the positions of Assistant Engineer and Chief Draftsman. In the summer of 1902, he became Assistant Bridge Engineer of the Louisville Bridge and Iron Company, in Louisville, Ky., where he remained about one year. During that time, he gave special attention to the design of bridges and steel structures. In 1903, he returned to Nashville as Bridge Engineer of the Nashville, Chattanooga & St. Louis Railway and filled that position until 1907.

From 1907 to 1913, Mr. Jones was engaged in private practice, as a Consulting Engineer, in Nashville. Although his practice included investigations and reports in other fields of engineering, he specialized in the design of steel and concrete structures, and his reputation as a Bridge Engineer, during this time, became well established. All his work was done with a thoroughness and ability that won for him the confidence of clients, contractors, and associates.

In 1908 and 1909, as Engineer for the Cumberland River Bridge Commission, Mr. Jones was charged with the design and erection of two important steel and concrete bridges over the Cumberland River, at Nashville. The structures were built at a cost of $1 000 000 and constitute an enduring monument to his skill in the design, and his ability in the direction, of engineering works of magnitude.

In 1912, Mr. Jones was employed by the Niagara Alkali Company, as Consulting Engineer, in the design and construction of a large chemical plant of reinforced concrete, at Niagara Falls, N. Y. The approbation which rewarded his labors in this work, on the part of both the owners and the contractors, was indicative of his success in the profession.

In 1913, pursuant to an amendment of the Act to regulate commerce, known as the Valuation Act, the Interstate Commerce Commission appointed a Board of Engineers as the nucleus of an organization to make a valuation of all interstate common carriers in the United States. The Board was composed of five engineers, appointed from five districts into which the country was divided. Mr. Jones was the appointee from the Southeastern States and, with his four colleagues, entered actively into the work to which the last ten years of his professional life was dedicated. As a member of the Engineering Board, he brought to this task a knowledge of engineering practice, an acquaintance with railroad construction, and a capacity for comprehensive investigation, that admirably fitted him for the responsibility that devolved on this important body. He was a faithful and influential member at all board meetings held to formulate the methods and create the personnel by which the work was to be accomplished. Under his direction, the inventories and engineering reports for the Southern District were prepared, comprising about 50 000 miles of steam railroads and the corresponding lines and properties of the Western Union and Postal Telegraph Systems.

In 1921, when the field work in the five districts was finished, the Engineering Board and the district organizations were abolished. Mr. Jones was then made Supervising Engineer in charge of the completion of the engineering work for all districts, with headquarters in Washington, D. C., and had almost finished the task when his unexpected death occurred. On March 31, 1924, he was attending a hearing in the valuation of the New York, Philadelphia and Norfolk Railroad, in the offices of the Commission in Washington, when he was suddenly stricken and died within a few minutes. His death occurred in the prime of his life and at the zenith of his career as an engineer.

Howard Jones had a love for his profession and a zeal for his work that mark the successful man in all walks of life. He had a keen mind, a high purpose, and a loyalty to principle, which are distinctly attributes of the Engineering Profession; but he had also a judicial sense, not commonly possessed by engineers, which enabled him to penetrate some of the intricacies of the law in its application to engineering problems. He was a rapid reader, with a retentive memory, and a taste for literature and history, as well as scientific knowledge.

On June 8, 1899, Mr. Jones was married, in Nashville, to Miss Marion Woodall Tucker, of Kansas City, Mo., who, with four daughters, survives him.

He was an exemplary citizen, a genial companion, and a good husband and father. He gave freely of his time and talents to progressive movements, whether professional, political, or social.

For many years, the Engineering Association of the South, of which he was a Past-President, owed its usefulness and activity in a large measure to his persistent efforts. At the time of his death, he was a member of the Cosmos Club, in Washington, and was President of the In-Com-Co Club of the Interstate Commerce Commission. He was also an active member of the Church of the Disciples.

Mr. Jones was elected a Member of the American Society of Civil Engineers on June 1, 1909.

HIRAM ALLEN MILLER, M. Am. Soc. C. E.*

DIED NOVEMBER 2, 1923.

Engineer, economist, and Christian gentleman, Hiram Allen Miller represented the highest ideals of professional engineering practice. After his graduation from Sheffield Scientific School at Yale University, he steadily rose by his unobtrusive yet keen initiative through the various steps of experience to a prominent place as consulting engineer in hydraulic and water-works construction. During his engineering activities, he had charge of the design and construction of public works the value of which was about $25 000 000.

Hiram Allen Miller was born at Williston, Vt., on June 3, 1853. He was the son of Charles Eliot and Emily (Clark) Miller. His ancestor, John Miller, who, with his father, Daniel, came from England not long after the arrival

* Memoir prepared by Sanford E. Thompson, M. Am. Soc. C. E.

of the *Mayflower,* was killed by the Indians at Springfield, Mass., in 1657. On his mother's side, also, the line of descent in Massachusetts goes back to early Colonial days.

Working on the farm in summer and attending the district school in winter, and after that a country academy, Mr. Miller laid the foundation for his persevering and industrious career. He entered Yale University with the Class of 1875, but left at the end of the first term because of the death of his father. He returned the following year in the Class of 1876, and entered Sheffield Scientific School, taking the course in Civil Engineering. He received prizes for excellence in mathematics, physics, and other studies in his Freshman year, and was Vice-President of his Class. He took a divided prize in mathematics in his Junior year. Later, he was President of his Class and also of the Christian Union. After his graduation in 1876 with the degree of Ph. B., he returned for a year for post-graduate work and also served as Instructor in Mathematics. In 1896, he was awarded the degree of Civil Engineer by the Sheffield Scientific School.

Like so many able men in the Civil Engineering Profession, Mr. Miller's earlier years present a panorama of frequent changes from the completion of one job to the commencement of another. Leaving Yale, he took up railroad engineering and from 1878 to 1884 was located in Southern Iowa, Nevada, Texas, and Louisiana. In 1880, following two years of railroad construction in the West, he became Chief Draftsman of the American Railway Improvement Company, then building the New Orleans Pacific Railway, and rose quickly to the position of Bridge Engineer and then Division Engineer in charge of 150 miles of construction. Later, he served as Division Engineer of the New Orleans and Mississippi Valley Railroad, including location and the difficult construction of the levee around the Bonnet Carre Crevasse. In 1884, he was Division Engineer of the New Orleans Pacific Division of the Missouri Pacific Railroad. His railroad experience involved responsible charge of about 300 miles of steam railroads and some electric lines.

From 1884 to 1892, Mr. Miller was located in Des Moines, Iowa. During these earlier years, also, he made excursions for brief periods into teaching and life insurance. In 1889, he was Chief Engineer on the location and construction of the Des Moines Belt Line Railway. In 1892, he accepted a position as Assistant Engineer on the construction of the Main Channel of the Chicago Drainage Canal, in charge of seven miles of construction. Three years later, in October, 1895, he came to Massachusetts and became Engineer in charge of the Reservoir Department of the Metropolitan Water-Works then under construction, directing the work of construction of the Wachusett Reservoir which has a capacity of 65 000 000 000 gal., at Clinton, Boylston, and West Boylston. He designed and built, subject to the approval of the Chief Engineer, the late Frederic P. Stearns, Past-President, Am. Soc. C. E., the North Dike of this reservoir, an earth dam of unique design nearly 100 ft. high in its maximum section. The work also involved forestry work from the planting of the seed in the seed beds to the final planting in the field, the improvement of the old forests, and the construction and maintenance of roads.

Then came the period of service from 1903 to 1910, as Chief Engineer of the Charles River Basin Commission, in charge of the construction of the Charles River Basin. This work required not only engineering supervision in construction, but careful hydraulic studies, and covered the design and construction of two locks, large sluices for flood flow, a draw-bridge, 85 ft. wide, with two street-car tracks, two large marginal conduits of the nature of intercepting sewers, a complicated electrical distribution plant, a steam distribution plant, and other features. The larger of the two locks was 45 ft. wide and 450 ft. long in the clear, with 18 ft. in clear depth below mean low tide.

This improvement transformed the Back Bay District of Boston, Mass., and the Cambridge water-front from areas contiguous to an ill-smelling tidal basin—extending with its muddy banks to Watertown and Newton—to beautiful residential and park sections facing a pleasing reservoir of constant level, suitable for boating and other water sports.

After the completion of the Charles River Dam in 1910, Mr. Miller opened an office for private practice as Consulting Engineer in Boston, and from that time until his death, he served various private projects and many municipalities, including Dalton, Lenox, Needham, Pittsfield, and Weymouth, in Massachusetts; New Britain and Bristol, in Connecticut; and Rutland and Vergennes, in Vermont. He also was called in for expert testimony on a number of damage suits and for the taking of land and water by various cities. His work at Pittsfield, Mass., included a masonry dam for water supply, about 100 ft. high, the cost of which totaled about $800 000.

As a contribution to the World War, he served as Resident Engineer, District No. 1 (New England), for the Emergency Fleet of the United States Shipping Board.

To recount his engineering services is to give only the professional side of his life. By his integrity, his quiet keenness, and his lovable character, Mr. Miller made many life-long friends and won a place of respect and honor in the community. In his church at Newton Highlands, Mass., he was completing at the time of his death twelve years' service as Deacon. He was a lifelong Democrat, not because of parentage, but because, as a student of economics, he believed in the fundamental principles of free trade. As an economist of the highest rank, the late President Wilson also appealed to his ideals.

Two years before his death, Mr. Miller had had a major intestinal operation. He had never fully recovered his health, and his death on November 2, 1923, from nervous collapse was the indirect result of this operation. He was buried in Williston Cemetery, at Williston, Vt.

Mr. Miller was married on October 7, 1884, to Martha A. Buckingham, of New Milford, Conn., who died on July 4, 1914. His second wife, to whom he was married on December 26, 1917, was Bertha Marion Converse, of Newton Highlands, the daughter of Martha and Conrad R. Converse. Of his first marriage, three children survive. The daughter, Anna Irene Miller, is Assistant Professor of English Literature at Goucher College, Baltimore, Md. One of the sons, Buckingham Miller was graduated, like his father, from the Sheffield Scientific School of Yale University in 1910 and is an engineer in New York City. Hiram Allen Miller, Jr., a graduate of the Classical School of

Yale University, 1917, is also an engineer engaged in industrial management for a large clothing plant in Chicago, Ill. Both sons served as commissioned officers in the World War.

Mr. Miller was a member of the Boston Society of Civil Engineers, Western Society of Engineers, American Water Works Association, New England Water Works Association, American Forestry Association, Massachusetts Highway Association, New England Historical Genealogical Society, American Economics Association, American Academy of Political and Social Science, Boston Chamber of Commerce, Congregational Club, St. Botolph Club, and Yale Club of Boston. He was a contributor to the *Journal* of the New England Water Works Association, the *Engineering News-Record,* and other technical periodicals.

Mr. Miller was elected a Member of the American Society of Civil Engineers on May 6, 1896.

ARCHIBALD OLIN POWELL, M. Am. Soc. C. E.*

Died November 18, 1923.

Archibald Olin Powell, the son of Pardon Archibald and Sarah Woodruff Powell, was born in Milwaukee, Wis., on August 31, 1859. He received his early education in the public schools of Milwaukee, and this was followed by a collegiate course in Engineering at the University of Wisconsin, from which he was graduated in 1882 with the degree of Bachelor in Civil Engineering. In 1890, he received the degree of Civil Engineer from the same institution.

Commencing as early as the summer of 1875, and continuing through the succeeding summers of his college vacations, Mr. Powell served as Recorder on U. S. Lake and Mississippi River Surveys and as U. S. Sub-Overseer on Wisconsin River improvement work. Resuming this latter work, on his graduation, he was soon given charge of the operation of the canal connecting the Fox and Wisconsin Rivers and of the Upper Lock on the Fox River. From 1882 to 1906, he was connected with the U. S. Engineer Office at St. Paul, Minn.—most of the time as Principal Assistant to the Engineer Officer in charge of that District. During this period, he was concerned chiefly with hydraulic investigations and improvement operations on the Chippewa, St. Croix, Minnesota, and Mississippi Rivers, and had charge of the design and construction of many important works.

In 1906, Mr. Powell came to Seattle, Wash., to prepare plans for and to report on the Lake Washington Canal Project, designed to connect Lake Washington and Lake Union with Shilshole Bay on Puget Sound by a lock and about four miles of ship canal. This investigation was made for a private corporation and, although the consummation of the project was delayed a few years, it was ultimately carried through, under direction of the U. S. Engineer Office, in large measure following the plans outlined by Mr. Powell. On the completion of this engagement, Mr. Powell established a permanent office in

* Memoir prepared by Joseph Jacobs, M. Am. Soc. C. E.

Seattle for practice as Consulting Engineer specializing in hydraulics and more particularly in river and harbor work. This practice he continued until his entry into the military service during the World War. He was among the first of the civilian engineers to enter the Service, and among the last to leave, his connection with the military establishment extending from April, 1917, to July, 1920. After his discharge from the Army, he returned to Seattle to resume the practice of his profession in association with Joseph Jacobs, M. Am. Soc. C. E., under the firm name of Powell and Jacobs. This association continued until Col. Powell's death, which occurred suddenly and unexpectedly on November 18, 1923, as result of a cerebral hemorrhage.

He was twice married. On February 6, 1882, he was married to Miss Annit Hortense Maloy at Portage, Wis. His second marriage which took place in 1909, at Seattle, was with Mrs. Margaret Foster Powell, the widow of his elder brother, Brig.-Gen. Charles A. Powell, U. S. Army. Col. Powell had only two children of his own, two sons by his first wife, but it was his pleasure also to help rear and educate the six children of his second wife, all of whom received collegiate training. He is survived by these eight children, five sons and three daughters, Mrs. Powell having died a month after Col. Powell's death.

Col. Powell always evinced a keen interest and actively participated in civic affairs, serving frequently on important committees of worthy civic organizations, and contributing valuable reports to them. He was ardently patriotic and much interested in military affairs. During the Spanish-American War, he organized one of the Companies, and was commissioned Captain, in the Second Regiment, U. S. Volunteer Engineers. During the World War, he was almost continuously attached to the office of the Chief of Engineers, U. S. Army, at Washington, D. C. He retired from the service with the rank of Lieutenant Colonel of Engineers.

Col. Powell was a man of firm and positive character and opinions, and he always expressed these opinions with fairness and fearlessness. He was the soul of honor, a man of high ideals, an able engineer, and to those who enjoyed his intimate acquaintance, a most companionable and lovable character.

Among his fraternal and society affiliations were the following: American Legion, Association for the Advancement of Science, Canadian Institute of Engineers, Seattle Chamber of Commerce, Seattle Municipal League, Rainier Club, and Beta Theta Pi. He served one year as President of the Pacific Northwest Society of Engineers and one year as President of the Seattle Section of the Society.

Col. Powell was elected a Member of the American Society of Civil Engineers on March 2, 1898.

JAMES GEORGE ROSS, M. Am. Soc. C. E.*

Died July 20, 1922.

James George Ross was born at Bethlehem, Pa., on January 30, 1878. He was graduated from Lehigh University in the Class of 1900.

* Memoir prepared by F. H. Hilliard, Esq., Memphis, Tenn.

For several years after his graduation, Mr. Ross was engaged in mining work in Pennsylvania. His first work for the Government was on the construction of the Plaquemine Locks connecting the Mississippi River with the Bayou Plaquemine, at Plaquemine, La. After supervising the construction of gun emplacements and an ammunition magazine at Fort St. Philip, La., he was placed in charge of the construction of the jetties in Southwest Pass, at the mouth of the Mississippi River, one of the most important and extensive works of its kind undertaken by the Government.

In 1908, Mr. Ross accepted the position of Assistant Superintendent of Dredging Operations under the Mississippi River Commission and was placed in charge of floating plant and the repair yard for the fleet of large hydraulic dredges used in the maintenance of low-water channels in the Mississippi River from Cairo to the head of the Passes below New Orleans, La.

In 1913, he left the Government Service to accept the management of the sand and gravel plant at Memphis, Tenn., of the Missouri Portland Cement Company. He remained in charge of this work until 1917, when he entered the First Officers' Training Camp at Fort Oglethorpe, Ga. On the completion of his training, he was commissioned a Captain and assigned to the 306th Engineers, 81st Division, participating in all the battles in which that Division took part.

On his return to Memphis in 1919, Captain Ross opened an office as Consulting Engineer and, later, became Managing Partner of Hilliard and Ross, Incorporated, Engineering Contractors.

His most notable characteristics were his force of character in handling men and his thoroughness in mastering the details of work. These characteristics are illustrated by the following incident in connection with his work on the Mississippi Jetties. On one occasion, the contractor suspended work on account of rough weather, stating that waves were running too high to attempt to tow the timber mattress, which had just been completed, into position and sink it. Captain Ross told the contractor that if he would let him have the tug and crew, he would tow the mattress into position and sink it, which he did, thus demonstrating to the contractor that the work could be carried on under the then existing weather conditions and speeding up the progress very materially.

While in the full vigor of an active career Captain Ross met an untimely death on July 20, 1922, by stepping into the path of a moving train.

Captain Ross was elected a Member of the American Society of Civil Engineers on May 28, 1912.

WILLIAM HORATIO SANDERS, M. Am. Soc. C. E.*

Died February 10, 1924.

William Horatio Sanders was born in Linlithgow, Scotland, on June 12, 1839. He was the youngest and only son of nine children. His mother died

* Memoir prepared by J. H. Quinton, M. Am. Soc. C. E.

in Edinburgh, when he was still a little child, his father having moved to that city soon after his birth.

About 1850, Mr. Sanders, the elder, with his son and two daughters came to America in a sailing vessel, being more than a month on the water. He opened a leather store in New York City, where his son's education was completed.

William Horatio Sanders entered the Army of the United States on August 7, 1862, and was honorably discharged on February 25, 1866, with the rank of First Lieutenant and Adjutant. He commenced the study of engineering about this time, and was employed in various engineering work until 1875.

From 1875 until 1887, he designed and constructed the dam and canal for a flour mill on the Pomme de Terre River, Minnesota. He reconstructed the Meeker (masonry) Dam, on the Chippewa River, Minnesota, and did much other work in connection with drainage and power plants in that State.

In 1887, Mr. Sanders moved to Southern California and devoted himself exclusively to hydraulic engineering. He designed and constructed the Mountain View Tunnel and the concrete pipe line from the tunnel at the mouth of San Antonio Canyon to the Mountain View Tract, Upland, Calif. On this line, he constructed a concrete siphon, which was said to be the first in California. He also designed and built the Fleming and Becket Tunnel and concrete pipe conduits, now known as the Indian Hill Tunnel and pipe line, near Claremont, Calif.

Mr. Sanders reconstructed the dam for the South Riverside and Temescal Water Companies, and was Engineer in Charge of Design and Construction for the well system at Lordsburg, Calif., and the concrete line from these wells to Covina, Calif., a distance of eight miles. He designed and built the sewer system for the City of Pomona, Calif., as well as the outfall sewer and septic tank, which was the first application of the septic process in California.

He also designed and constructed the following concrete or masonry reservoirs: Currier, Richards, Loop and Meserve, Packard Vineyard Tract, Evergreen Tract, and the concrete-covered reservoir for the water supply for the City of Pomona, together with the concrete pipe lines for these reservoirs, which are all in Los Angeles County.

The "Historical San Antonio Power Plant", the first with long-distance transmission, having a power line extending to Pomona and San Bernardino, Calif., was designed and constructed by him, as well as the Ontario Power Plant. There are nine tunnels on this supply pipe line. He also built the transmission line to Upland and Ontario, Calif.

For thirteen years, Mr. Sanders served as Chief Engineer for the Pomona Land and Water Company of Pomona, one of the largest water companies of Southern California, for which he had charge of all design and construction.

He was one of the Consulting Engineers for the United States Reclamation Service from May 1, 1903, until about 1918, and was connected with the design and construction of many of the large works of the Service, as well as those of the United States Indian Irrigation Service.

In the course of his engineering practice, Mr. Sanders designed and built many smaller domestic, irrigation, water and sewer systems, in various cities and towns of Southern California.

He was married twice, his first wife having been Miss Lorinda J. Fields. They had four children, who are still living: Mrs. Minnie Cronkhite, of Wapato, Wash., Mrs. Isabel Dibble, of Cannon Falls, Minn., William Francis Sanders, of Visalia, Calif., and Charles Horatio Sanders, of Los Angeles, Calif. Mrs. Sanders died on December 3, 1875. On September 19, 1878, he was married to Miss Emma Adelia Ellsworth, an accomplished lady with literary tastes, who took an interest in all his work, and was a real help to him in much of it. She survives him. Their married life of more than 45 years was ideal, and constitutes a good example for the rising generation.

Mr. Sanders was a Mason for more than 60 years and was a great admirer of that Order. He took nearly all the degrees, was a Scottish Rite, Knights Templar, and Shriner of Al Malaikah Temple of Los Angeles. He was also a Member of the Military Order of the Loyal Legion of the United States, and a charter member of the Benevolent Protective Order of Elks, of Pomona.

He was a man of sterling character,—kindly, considerate, and polite. Notwithstanding his military training, he was always a peacemaker, and had many of the best traits of his Scottish ancestry. He had a remarkably upright and dignified bearing, strictly in keeping with his character. He was an honorable, upright Christian gentleman, and is greatly mourned by his old friends in Southern California.

Mr. Sanders was elected a Member of the American Society of Civil Engineers on February 3, 1904.

CLINT SANFORD SLAYBACK, M. Am. Soc. C. E.*

Died October 26, 1922.

Clint Sanford Slayback was born at Princeton, Ind., on February 11, 1859, and attended the public schools of that city until he was sixteen years of age.

In 1876, he entered the employ of the Cincinnati Southern Railroad Company, as Axeman and Rodman. After two years of service with this Company, he accepted the position of Rodman and Instrumentman with the Louisville, New Albany and St. Louis Air Line, now the Southern Railway Company. From 1880 to 1881, he served as Resident Engineer of the Mt. Vernon Branch of the same railroad, and, during 1882, held the position of City Engineer of Mt. Vernon, Ind.

From 1882 to 1885, Mr. Slayback was Resident Engineer of the Evansville and Indianapolis Railroad, and, later, accepted the position of Engineer with the Evansville and Terre Haute Railroad Company, which he retained until 1887, when he entered private practice as a Consulting Engineer. In 1888, he became Resident Engineer of the Evansville and Richmond Railroad, and served in this capacity for three years, when he was made City Engineer of Anderson, Ind. From 1893 to 1897, he was again engaged in private practice as Consulting Engineer.

* Memoir prepared by J. H. Brillhart, M. Am. Soc. C. E.

Mr. Slayback then entered the paving business as Engineer and Superintendent for the Talbot Paving Company at Shreveport, La. He held this position until 1898, doing pioneer work in paving, after which he accepted the appointment of Locating Engineer with the Northern Railroad Company, between Little Rock, Ark., and Springfield, Mo. In 1899, Mr. Slayback was made Engineer in Charge of the Chicago, Portland, and Southeastern Railroad and was engaged in promotion work, holding this position for one year. From 1900 to 1901, he served as Chief Engineer for the Searcy and Des Arc Railroad Company.

His next engagement was that of Resident and Locating Engineer of the Missouri, Kansas and Texas Railroad System, which position he resigned in 1903 to re-enter private practice. From 1905 to 1906, he acted as Chief Engineer for the White River and Mountain Home Railroad Company and from 1906 to 1911, as Office Engineer for the Union Railway Company, at Memphis, Tenn. During 1912, he resumed his private practice.

In 1913, Mr. Slayback was appointed Chief Engineer of the McNerney Company of Muskogee, Okla., but, in 1915 and 1916, was again engaged in private practice. In 1917, he became Engineer of the Talbot Engineering Company at Houston, Tex., and remained with this firm for one year, afterward accepting a position with the Engineering Department of the Texas Company of Houston and Cisco, Tex., in the Refinery and Production Departments. He was a popular and agreeable companion, and well liked by his co-workers.

Mr. Slayback was elected a Member of the American Society of Civil Engineers on March 7, 1921.

FREDERICK PUTNAM SPALDING, M. Am. Soc. C. E.*

Died September 4, 1923.

Frederick Putnam Spalding, the son of Israel Putnam and Ruth Elizabeth (Cooley) Spalding, was born at Wysox, Pa., on April 7, 1857. He was of English ancestry, a descendant of Edward Spalding, who first settled at James Cittie, Va., in 1619 and removed to Braintree, Mass., in 1630. It is of interest to note that in 1663 Edward Spalding was "the Surveyor of Highways" at Chelmsford, Mass. Israel Putnam Spalding served as a Union soldier during the Civil War and participated in the Battles of Fredericksburg, Chancellorsville, and Gettysburg. He was mortally wounded at Gettysburg on July 2, 1863, and died twenty-six days later, when his son, Frederick, was only six years of age.

Frederick Putnam Spalding received his preliminary education in the public schools of Towanda, Pa. He then entered Lehigh University and was graduated with the Class of 1880 with the degree of Civil Engineer.

Immediately after his graduation, Mr. Spalding entered the employ of the Southern Pacific Railway Company as Levelman on preliminary and location

* Memoir prepared by A. Lincoln Hyde, M. Am. Soc. C. E.

surveys in Texas and New Mexico, resigning this position in June, 1882, to become Recorder and Observer on preliminary triangulation on the U. S. Geodetic Survey in the State of Pennsylvania. He remained at this work, however, only four months.

From October to December, 1882, he served as Transitman on the survey of Memphis Reach, Mississippi River; from January to March, 1883, he was in charge of hydraulic grading at Memphis Reach; and from July, 1883, to March, 1884, he was Assistant to the Engineer in charge of that work.

From April to August, 1884, Mr. Spalding was Assistant Engineer in charge of a field party on a survey of the Arkansas River, and from August, 1884, to March, 1885, he was Assistant to the Engineer in Charge, in direct charge of all the work of Memphis Reach and Harbor.

From February, 1885, until August, 1886, Mr. Spalding was engaged in private practice in Memphis, Tenn., under the firm name of Spalding and Burrowes. He was then called to his Alma Mater, Lehigh University, as Instructor in Civil Engineering, but resigned in October, 1888, to become Engineer in Charge of Surveys for the Third District of the Lower Mississippi, at Greenville, Miss. He relinquished this work in June, 1890, to become Engineer in Charge of Street Extensions in the District of Columbia, and in November, 1890, he was appointed Engineer in Charge of Rock Creek Park, Washington, D. C.

In October, 1891, Mr. Spalding was called to Cornell University, Ithaca, N. Y., as Assistant Professor of Civil Engineering and continued in this position for seven years, when he resigned to engage in business as a Contractor on municipal work in Bethlehem, Pa. He remained in this line of work only two years.

In September, 1900, he went to the University of Missouri at Columbia, Mo., as Professor of Civil Engineering, and during the academic year 1905–06, he served as Junior Dean of the School of Engineering. He continued as Professor of Civil Engineering in the University of Missouri until his death on September 4, 1923.

Professor Spalding was married on January 22, 1885, to Miss Annie Packer Wilbur, of Bethlehem, Pa. He is survived by his widow and two daughters, Marion Elizabeth and Aurelia, and by a brother, Israel Putnam Spalding, Jr., of Towanda.

Professor Spalding was a Fellow of the American Association for the Advancement of Science, a member of the Society for Testing Materials, the Society for the Promotion of Engineering Education, the Society of Sigma Xi, Tau Beta Pi, and the Faculty Club of Columbia, Mo.

He was the author of "Notes on Hydraulic Cement" (1893); "Text-Book on Roads and Pavements" (1894, 1901, 1908); "Hydraulic Cement, Its Properties, Testing, and Use" (1897, 1905); and "Masonry Structures" (1921).

While on his way back from a vacation pleasure trip to Southern California with his wife and two daughters, Professor Spalding was seized with an attack of heart failure in the railway station at San Francisco. The party was able to continue its trip, but he suffered a second attack at Portland, Ore. Following medical advice and accompanied by a trained nurse, the family con-

tinued on the journey home, but at St. Paul, Minn., it became necessary to remove Professor Spalding to St. Luke's Hospital. He rallied again, but died suddenly as he was chatting with his nurse about twelve hours after his arrival at St. Paul. He was buried at Bethlehem, Pa.

The following resolution was adopted by the Faculty of the School of Engineering of the University of Missouri:

"Frederick Putnam Spalding, for twenty-three years Professor of Civil Engineering in the University of Missouri, died suddenly just before the opening of the Fall Semester, 1923. In his death we have suffered the loss of one who was a recognized authority in his special field, a wise and patient teacher, a valued counselor to this faculty, a just, gentle, and beloved man. His memory will remain with us his colleagues, as it will with those who have gone out from his teaching into the engineering work of the world—an inspiration and a blessing."

Professor Spalding was elected a Member of the American Society of Civil Engineers on December 2, 1891.

HENRY HAYES WADSWORTH, M. Am. Soc. C. E.*

DIED JULY 7, 1923.

Henry Hayes Wadsworth was born on January 13, 1865, at New Haven, Conn., in which State his forebears had resided for many generations. The founder of the Wadsworth family in America, William Wadsworth, landed at Salem, Mass., in 1632, from England, and soon afterward settled at Hartford, Conn. His son, Captain Joseph Wadsworth, was the chief actor in that well known story of the concealment, in the now famous Charter Oak, of the charter of the Connecticut Colony, when it had been demanded for revocation by Sir Edmund Andros, first Governor-General of New England. Henry Hayes Wadsworth was of the seventh generation in direct line from Joseph Wadsworth and of the eighth generation from the original settler. His mother, who was Fidelia Munn Gilbert, was the descendant of a long line of Gilberts who had lived in Connecticut since some time prior to 1660, when the first of the family served as Deputy Governor of the New Haven Colony.

In 1870, his father, Henry Wadsworth, moved to Minnesota with his family of three boys and one girl, and established a new home at Glencoe, a short distance west of Minneapolis. Here, Henry Hayes Wadsworth, the second son, received his common school education. Later, however, he returned to New Haven to take his college preparatory work and to attend the Sheffield Scientific School of Yale University. He completed the regular course in Civil Engineering, and was graduated with special honors in 1886, at the head of the Civil Engineering Section of his class.

After his graduation, Mr. Wadsworth returned to Minnesota, arriving at a time when pioneer railroad work was in full swing. For four years, he was engaged on the location and construction, in Minnesota and North Dakota, of lines which afterward became parts of the Great Northern System. The latter

* Memoir prepared by Lee S. Griswold and Ralph G. Wadsworth, Associate Members, Am. Soc. C. E.

portion of this period he served as Assistant Engineer and Chief Draftsman of the Eastern Railway of Minnesota, during which time the elevated sections of that road in Duluth, Minn., were designed and built. This work included many special structures and complicated crossings. He then moved to the head of Lake Superior, where a rapidly developing harbor offered promise of a greater breadth and variety of opportunity. From 1891 until 1895, he was Principal Assistant City Engineer of Superior, Wis., in charge of all street, bridge, sewer, and harbor works. Among other improvements under his direction, about 30 miles of pavements were laid and 40 miles of sewers built. Following his municipal work at Superior, he was engaged by the Lake Superior and Ishpeming Railway Company, as Acting Chief Engineer. For about one year he had charge of the revision of its location and the building of its road from Ishpeming to Marquette, Mich., together with the design and construction of ore and merchandise docks for the railroad, in the harbor at Marquette.

In September, 1896, Mr. Wadsworth reported for duty at the U. S. Engineer Office in Duluth, which office has supervision of the improvement of the Duluth-Superior and other harbors on Lake Superior, and thus (except a previous period of temporary employment during the summer of 1895) he began his service with the United States Government, which lasted for more than twenty-three years. Soon after his appointment, he was promoted to the grade of U. S. Assistant Engineer and placed in immediate charge, at Superior, of dredging operations involving the removal of approximately 21 000 000 cu. yd. of material, about equally divided between the harbors at Duluth and Superior, carried on under a contract that continued for about five years. In addition to the dredging referred to, harbor developments under way included the dredging of other channels and basins, establishing beacons, locating docks, and the construction of breakwaters and jetties, together with the hydrographic surveys incident thereto. The harbor surveys were largely made in winter, when ordinary land-surveying methods could be used on the ice, soundings being taken through borings made by specially designed portable machines. On the dredging work, both hydraulic and dipper dredges of large capacity were used. Mr. Wadsworth was in direct control of much of this work. He also had charge of extensive additions to the training walls of the Duluth Ship Canal and prepared one of the early plans for the aerial bridge which was constructed later across the Canal.

In 1905, Mr. Wadsworth accepted a transfer to San Francisco, Calif., where he was appointed Principal Assistant Engineer for the California Débris Commission and Second San Francisco District Engineer Office. The Débris Commission is a Board of Army Engineer Officers which has control of hydraulic mining operations in California. The work of this Board has been described in a paper* presented to the Society by William W. Harts, Colonel, Corps of Engineers, U. S. Army, M. Am. Soc. C. E., entitled, "The Control of Hydraulic Mining in California by the Federal Government." At the time of Mr. Wadsworth's arrival, a contract had just been let for the raising of the Yuba River Barrier, a restraining weir which formed one unit of the extensive project for river improvement described in the paper mentioned.

* *Transactions*, Am. Soc. C. E., Vol. LVII (1906), p. 1.

Work on the Daguerre Point Cut and Settling Basin, the principal other feature of the project, was also under way. Supervision of the work on these and other portions of the plan formed a large part of Mr. Wadsworth's duties. The work done in connection with the Barrier, until its failure in 1907, was described by him in a paper* entitled "The Failure of the Yuba River Débris Barrier and the Efforts Made for Its Maintenance", presented to the Society in 1910.

The Secretary and Disbursing Officer of the Débris Commission serves also as District Engineer in charge of the control and improvement of the Sacramento and the San Joaquin Rivers with their tributaries in the central valley of California. Aside from actual construction work in connection with the restraint of mining débris, Mr. Wadsworth's duties included a wide range of studies and inspections throughout the central part of the State. Hydraulic mine inspections led to many remote places on the numerous tributaries of the Sacramento River and gave him a comprehensive grasp of stream-flow conditions in the Sierra Nevada Mountains. Surveys and examinations of the lower reaches of the rivers, to determine the effect of mining operations on navigation and agriculture, gave him an equally broad knowledge of valley conditions. Thus, he became eminently fitted to take up the problem of flood control of the Sacramento and San Joaquin Rivers, which problem occupied most of his attention during the period from 1908 to 1919.

In 1908, surveys were made under his immediate direction of the Sacramento River from Suisun Bay to the mouth of the Feather River and of the San Joaquin to Stockton, Calif. The resulting maps on a scale of 400 ft. to the inch covered more than 120 miles of river and included frequent cross-sections and a wide strip of adjacent topography. In 1909, the Sacramento River survey was extended about 120 miles farther up stream.† These extensive surveys were completed at about the same time that the first accurate measurements of severe floods on the Sacramento became available. In 1907 and 1909, floods occurred of greater magnitude than had ever previously been measured and they at once furnished the fundamental quantities on which studies of flood control had thereafter to be based. With these quantities in mind and the recently completed surveys before him, Mr. Wadsworth began studies looking to the formulation of a comprehensive flood-control plan for the Sacramento Valley. He soon demonstrated the unfeasibility of the channel enlargement plan, which had formerly been recommended, and then proceeded to a detailed design of a by-pass project, providing for the relief of the whole valley. His report, dated July 14, 1910, was embodied in the report of the California Débris Commission which was approved by the U. S. War Department and which has been the basis for the rapid and continuous development of the project since that time.

Immediately after the completion of his flood-control report, Mr. Wadsworth was transferred to the Department of the Interior and began an exhaustive study of all possible sources of water supply for the City of San Fran-

* *Transactions*, Am. Soc. C. E., Vol. LXXI (1911), p. 217.

† These surveys were published as Documents Nos. 1123 and 1124, 60th Cong. 2d Sess., and Document No. 76, 62d Cong. 1st Sess.

cisco, with a view to making recommendations as to granting to the City certain rights in the Hetch Hetchy Valley and Yosemite National Park. This investigation consumed a period of two and one-half years and resulted in the accumulation of a great mass of information, which was completely summarized and correlated in a report presented to the Advisory Board of Army Engineers in January, 1913.

Returning to duty with the War Department, a survey of the San Joaquin River, above Stockton, was made during the next two years, extending as far as Herndon, Fresno County, and including about 200 miles of river. The main purpose of this survey was to establish a basis on which plans for the control of floods in that valley could be developed, and partly to investigate the possibility of navigation of the river. The following years were occupied with studies and surveys of various phases of the flood-control projects and with certain construction work which had been undertaken by the Federal Government.

In the spring of 1917, Mr. Wadsworth applied for and received a commission as Major of Engineers in the U. S. Reserve Corps and, in May, spent eleven days at the First Officers' Training Camp at the Presidio of San Francisco, before being relieved to resume his more pressing civil duties with the Engineer Department. In July, 1917, he was again on active duty for a short period at Salt Lake City, Utah, where he served as Senior Officer on a Board appointed to examine applicants for commissions in the Engineer Reserve Corps.

In January, 1920, Mr. Wadsworth resigned from the Government service to open an office in San Francisco, for consulting and general engineering practice, specializing in works for the utilization of water and for protection against floods. The announcement of his entrance into the ranks of consulting engineers brought immediate response from many quarters. He was engaged to make numerous examinations and reports on dams, irrigation and water supply projects, power development schemes, reclamation works, hydraulic mine projects, and was retained in the famous Antioch suit, which was directed against practically all the water users in the central valley of California. During this period, he also managed to devote a large amount of time to activities outside his engineering practice. As Chairman of the Section on Harbor Control of the Commonwealth Club of California—a public welfare organization of State-wide extent—he gave a great deal of effort to investigations and reports on various matters pertaining to the commercial development of San Francisco Bay and Harbor. He also served as Chairman of a Committee which reported to the Oakland, Calif., Chamber of Commerce on the water supply for the cities on the eastern side of San Francisco Bay just prior to the formation of the East Bay Public Utility District.

Mr. Wadsworth died in San Francisco on July 7, 1923, after an illness of only a few months. He is survived by two sons, Ralph G. Wadsworth, Assoc. M. Am. Soc. C. E., and Harold A. Wadsworth, Assistant Professor in the Department of Agriculture, University of California. His wife, who was Jennie M. Anderson, of North Dakota, and formerly of Ontario, Canada, died in 1919.

Mr. Wadsworth was a member of the Engineers' Club of San Francisco, the Commonwealth Club of California, the Home Club, of Oakland, Calif., the American Legion, the Society of American Military Engineers, and the Pacific Association of Consulting Engineers. He was a frequent attendant at the First Congregational Church of Oakland, the city in which he had made his home from the time of his first arrival in California.

He was of a modest but genial and kindly disposition, ever courteous and patient, yet energetic and forceful in completing whatever was undertaken, thorough, eminently just, possessed of high ideals, and of absolute integrity. He delighted in association with his friends and acquaintances, both professional and social, and held the respect, esteem, and confidence of all who knew him.

Mr. Wadsworth was elected a Member of the American Society of Civil Engineers on October 2, 1901. He took an active interest in the Society at all times and in addition to the paper previously mentioned, contributed numerous discussions on a wide variety of subjects to its *Transactions*. At the time of his death, he was Vice-President of the San Francisco Section.

FREDERICK CHARLES YOUNG, M. Am. Soc. C. E.*

DIED MARCH 18, 1924.

Frederick Charles Young, the son of George N. and Mary Hunter Young, was born in Woodbine, Iowa, on June 13, 1886. He was prepared for college at the Woodbine Normal School, from which he went as honor scholar to the College of Applied Science of the State University of Iowa in the fall of 1906. He was graduated in June, 1909, completing the four-year course in three years, earning a large part of his own support, and winning three marked distinctions, namely, election to the Society of Sigma Xi, and Tau Beta Pi, Honorary Engineering Society, and the Cox Prize of $100 for the best graduating thesis. On graduation, he received the degree, Bachelor of Engineering, and in 1913 was given the degree, Civil Engineer.

During his summer vacations Mr. Young engaged in engineering work and immediately after his graduation assumed the position of Assistant City Engineer of Iowa City, Iowa, in charge of the construction of sewers and paving.

From September, 1909, to February, 1914, Mr. Young was an Instructor in Civil Engineering in the State University of Iowa, engaging in engineering work during his summer vacations. In June, 1913, he was given a contract to build a tunnel, ½ mile long, for the heating system of the State University of Iowa. He was so successful with this work that he determined to engage in contracting, and left the University in February, 1914, to begin this occupation in which he continued until September, 1917, when he entered active service in the United States Army, having been commissioned a Captain in the Engineer Officers' Reserve Corps, on August 8, 1917.

* Memoir prepared by William G. Raymond, M. Am. Soc. C. E.

He reported for duty at the Second Engineer Officers' Training School, Fort Leavenworth, Kansas, on September 25, 1917. He was assigned to the 114th Engineers, 39th Division, Camp Beauregard, Louisiana, on December 10, 1917, and was appointed Regimental Adjutant, on March 2, 1918. He sailed for France on August 2, 1918, on the U. S. S. *Wilhelmina,* arriving at Brest on September 3. On October 3, he was placed in command of the First Battalion, 114th Engineers, and ordered to the front in the Meuse-Argonne engagement, where he was in charge of the maintenance of one of the three roads supplying this area until the signing of the Armistice. Captain Young sailed for the United States on April 1, 1919, on the U. S. S. *Nebraska,* arriving at Newport News, Va., on May 2, and was honorably discharged at Camp Dodge, Iowa, on May 30, 1919. The endorsement on his service record by the commanding officer was: "An engineer of marked ability, a careful administrator, and a capable executive".

He immediately resumed his contracting work with the Moore-Young Construction Company, in which his partner was Mr. Stanley D. Moore, of Waterloo, Iowa. The firm was engaged principally in the construction of municipal works, and Captain Young continued in this work until his death.

He was married to Helen S. Ordway on July 22, 1908, at Glens Falls, N. Y. Two children, twins, were born to them, one of whom, Frederick Charles, Junior, died in infancy. The other, Gerald Ordway, and Mrs. Young are living.

Although Captain Young was a man of most persistent energy and attention to duty, as indicated by the record of his college work, yet he found time in the latter years of his more than usually successful business life for recreation and participation in public affairs. He was a lover of horses, and he and his wife and son found pleasure and health in riding the fine horses that he owned.

He was a member of the Rotary Club of Waterloo, and just before his death had been selected by the Committee on Nomination for Eldership in the Westminster Presbyterian Church of Waterloo.

A progressive and able executive, seeking and adopting the most advanced approved methods of conducting work, absolutely honest and honorable in all his dealings, Captain Young was one among the very small group of men who are able to build successful businesses in a few years and win and retain the respect and even affection of all those who know them well.

Captain Young was elected an Associate Member of the American Society of Civil Engineers on February 4, 1914, and a Member on August 12, 1920.

JUAN BATISTE HIPOLYTE BARDURY, Assoc. M. Am. Soc. C. E.*

Died September 20, 1923.

Juan Batiste Hipolyte Bardury, was born at San Pierre, Martinique, on October 28, 1875. He received his technical education in France, where he

* Memoir prepared by John M. Adams, Assoc. M. Am. Soc. C. E.

was graduated in July, 1892, from the Rouan Professional School, with the degree "Contre Maitre."

Mr. Bardury followed various lines of work until September, 1896, when he was employed as Draftsman in the Compagnie des Mines at Chemin de Fer de la Guayanne Francaise. Six months later, he became Instrumentman in the same Company, which position he held until April, 1898. He then went to Havana, Cuba, and accepted a position with Poaling and Company, Limited, of London, England, as Inspector on the construction of the Malecon in the Harbor of Havana, which position he held until October, 1898. From that date, until April, 1899, he served as Private Secretary to the late General Frederick D. Grant, U. S. A., in Cuba.

From April, 1899, to July, 1902, Mr. Bardury was Draftsman and Instrumentman on the survey and construction of the new roads and bridges made by the Bureau of Public Works of Porto Rico, and, from August, 1902, until December, 1904, he held the position of Assistant Chief Engineer of the American Railroad Company of Porto Rico, on the survey and construction of the line from Mayagüez to Yauco.

From June, 1905, until March, 1906, he was engaged in the private practice of engineering in Porto Rico, during which time he drew the plans of the City of San German, Porto Rico, and constructed the market building for that city. From August, 1906, until February, 1907, Mr. Bardury served as Superintendent on the construction of 20 km. of railroad for the Fajardo Development Company, of Porto Rico, and from February to December, 1907, he was Assistant Superintendent on the construction of the Power Plant, Dam, and Tunnel at Comeria Falls, Porto Rico, for the J. G. White Company.

From January, 1908, to May, 1909, he was Assistant Engineer with the Bureau of Public Works, Porto Rico, in charge of the construction of the "Reyes Católicos" Bridge, a steel structure 100 m. long, and also of a 22-km. section of road construction from Lares to Yauco. From June until July, 1909, he was Draftsman in the Porto Rican Irrigation Service, and from July, 1909, to November, 1914, he held, successively, the positions of Assistant Engineer, Assistant Superintendent, and Superintendent of Construction, in the Porto Rican Irrigation Service on the surveys and construction of the Carite Dam and Tunnel.

From December, 1914, to the summer of 1915, Mr. Bardury visited his brother in Martinique. The brothers were the sole surviving members of their family, the others having been lost during the eruption of Mount Pelée in 1902.

In July, 1915, Mr. Bardury became Assistant Engineer with the Department of Public Works of the Dominican Republic, and was employed during the next five years on the location and construction of highways. In December, 1920, he was appointed District Engineer of the Southern District, which position he held at the time of his death.

Mr. Bardury was a man of sterling character and marked ability and showed an earnestness and energy in everything which he undertook that inspired the confidence and esteem of all with whom he came in contact.

He was married on March 2, 1918, to Señorita Gloria Sanchez who, with his brother, survives him.

Mr. Bardury was elected an Associate Member of the American Society of Civil Engineers on April 2, 1913.

WILLIAM GEORGE JUENGST, Assoc. M. Am. Soc. C. E.*

Died March 5, 1924.

William George Juengst was born on April 6, 1884, on a farm about four miles from North Vernon, Ind. He was the eldest son of Charles F. and Auguste (Schuster) Juengst, both natives of Germany. He attended the Grammar School near his father's farm and was graduated from the High School at North Vernon.

From September, 1904, to June, 1905, Mr. Juengst was engaged under H. F. Juengst and C. J. Eld, Jr., Members, Am. Soc. C. E., in East St. Louis, Ill., on the construction of a 4 000 000-gal. settling basin, extension of intakes, and other work. He then attended Purdue University, Lafayette, Ind., from 1905 to 1909 and was graduated as Bachelor of Science in Mechanical Engineering. During his vacations from June to September in 1905 and 1906, he worked on the construction of the purification plant and pumping station of the South Pittsburgh Water Company, under John N. Chester, M. Am. Soc. C. E., and Mr. Eld. In 1908 and 1909, he was employed as Draftsman by the American Water Works and Guarantee Company, Pittsburgh, Pa., under G. W. Biggs, Jr., M. Am. Soc. C. E. In August, 1909, he was sent to Birmingham, Ala., and acted as Assistant Engineer on the construction of Lake Purdy Dam, Rosedale Pump Station, and the improvement of the North Birmingham Pump Station, under Messrs. Juengst and Eld, until 1911. He was then made Engineer in charge of construction at the different plants of the American Water Works and Electric Company, at New Castle, Pa., Wichita, Kans., Joplin, Mo., St. Joseph, Mo., Chattanooga, Tenn., Birmingham, Ala., and Keokuk, Iowa. His work in these plants consisted of reconstructing pump stations and building filters, laying pipes, moving and erecting machinery, etc.

From 1913 to 1916, Mr. Juengst was Superintendent of the Keokuk Water-Works Company, at Keokuk, Iowa. He was then sent to East St. Louis, Ill., as Engineer in charge of the construction of an 8 000 000-gal., concrete filter plant, a 3 000 000-gal., covered clear-water and a 5 000 000-gal., sedimentation basin, and a modern coagulating plant.

After the completion of this work, Mr. Juengst had charge of construction work in Muncie, Ind., Wichita, Kans., and since 1921, at Huntington, W. Va., where he rebuilt the entire pump station, and constructed a sedimentation basin and concrete filter plant. His last work was the construction of a new low-service station on the Ohio River a mile above the mouth of the Guyan River. The work consisted of the construction of a pump pit 75 ft. deep, laying two

* Memoir prepared by George W. Biggs, Jr., M. Am. Soc. C. E.

intake pipes into the Ohio River and a force main across the Guyan River to the sedimentation basin at the high-service station, and the erection of two vertical motor-driven centrifugal pumps. This work was entirely finished when Mr. Juengst was taken ill with appendicitis, which resulted in his death.

Mr. Juengst was married on December 24, 1911, to Effie Jane Gray of North Vernon, Ind., who died at Oakdale, Iowa, in 1916. Of this union, there were three children, two girls and a boy; the latter died in 1917. He was married again in East St. Louis, Ill., in July, 1920, to Myrtle B. Lively, who, with his two daughters, survives him.

Of sterling character, Mr. Juengst was possessed of a quiet, pleasant disposition. He was a hard, industrious, conscientious worker and devoted to his family and parents.

He was especially fitted for construction work of large and difficult character. He was a good organizer, and was admired and respected by all who worked under him and were associated with him, as well as by all his superior officers.

Mr. Juengst was elected an Associate Member of the American Society of Civil Engineers on November 27, 1917.

SWAN AUGUST KALBERG, Assoc. M. Am. Soc. C. E.*

Died November 5, 1922.

Swan August Kalberg was born at Orsos, Sweden, on September 20, 1886. He came to the United States in his early childhood and was educated in the public schools of New Britain, Conn., having been graduated from the High School in 1904. Later, he entered Cornell University from which he was graduated in 1910 with the degree of Civil Engineer.

During the summer of 1908, Mr. Kalberg held the position of Steel Detailer with the American Bridge Company and with the East Berlin Construction Company. He resumed his studies at Cornell University in the fall of 1908, returning to his position with the Berlin Construction Company in June, 1909, where he remained until October. After his graduation, he was employed by this Company from July, 1910, to March, 1911.

In March 1911, he served for one year as Structural Designer for Stone and Webster Engineering Corporation at the main office in Boston, Mass., and was then transferred to Keokuk, Iowa, in connection with the construction of the hydro-electric power station at the Keokuk Dam. After his return to Boston Mr. Kalberg was engaged on the design of reinforced concrete and steel work for the same firm.

In 1914, he accepted the position of Reinforced Concrete Designer with the Aberthaw Construction Company and remained with that firm until July, 1915. He then entered the employ of the F. R. Ley Company at Springfield, Mass., as Structural Engineer in charge of design, his work including the design of many large undertakings of the Company during the World War. The

* Memoir prepared by S. M. Kalberg, Esq., Boston, Mass.

illness from which he never recovered came at the end of this strenuous period and resulted in his death on November 5, 1922.

Mr. Kalberg was elected an Associate Member of the American Society of Civil Engineers on November 27, 1917.

CLARK OLDS, Assoc. M. Am. Soc. C. E*

DIED AUGUST 16, 1922.

The death on August 16, 1922, of Clark Olds, Assoc. M. Am. Soc. C. E., brought to an end a kindly, honorable, and useful life. Starting as an engineer, he changed to the profession of law, became prominent in civic affairs, and left a record of conspicuous service to his community in furthering engineering works. His career illustrates the influence and value of an engineering training.

Clark Olds, the son of Lewis Wilson and Louisa Everett (Ackerly) Olds, was born near Erie, Pa., on July 14, 1850. His family removed from New England to Pennsylvania in pioneer days. After attending the public schools and Erie Academy, he entered the University of Michigan and was graduated therefrom as Bachelor of Science in 1870. He received his Master's Degree two years later.

During vacations and after his graduation, Mr. Olds was engaged with the U. S. Lake Survey. In 1872 he gave up this work and turned his attention to the study of law. After two years at the University of Leipsic in the study of Roman and international law, he returned to Erie, entered the law office of William Benson, Esq., and was admitted in 1876 to the Erie County Bar and subsequently to practice before State and Federal Courts. Thereafter, for nearly fifty years, Mr. Olds pursued a successful general practice, specializing in admiralty law.

He was married on December 13, 1876, to Livia Elizabeth Keator, of Cortland, N. Y. Of their three children, one survives, Irving Sands Olds, a graduate of Yale University and a successful attorney of New York City.

Mr. Olds was prominent in the development of the City of Erie, its waterworks, harbor, docks, and public parks and institutions, and the activities of its Chamber of Commerce. During his régime as Water Commissioner and President of the Board, the intake was extended, the pumping and distribution systems were reconstructed and enlarged, and a generous supply of good water was provided for the growing needs of the municipality with intelligence and foresight. The improvement became a factor in the location of manufacturing interests at Erie. As Chairman of the Harbor Commission, his efforts were largely responsible for securing the legislation and funds that resulted in much needed water-front improvements. Construction in this connection was under the active charge of his Commission and, on its completion, a substantial unexpended balance was returned to the State. Always interested in the development of the harbor and peninsula, his constant work in that direction

* Memoir prepared by Frederick C. Noble, M. Am. Soc. C. E.

has met with success and is expected to be followed by the extension of the Park System on a large scale. He also took active part in the raising of Perry's flagship, the *Niagara,* for the Centenary of the Battle of Lake Erie.

Mr. Olds filled many posts of honor and trust at various times. He was President of the Erie Chamber of Commerce and of the Erie County Mutual Fire Insurance Company and, at the time of his death, was a Director of the Erie Trust Company and of the L. W. Olds Real Estate Company and President of the Erie and Lakeside Cemeteries. For many years, he was a delegate to the International Peace Conference at Lake Mohonk, N. Y. During the World War, he served as Chairman of the Draft Questionnaire Board and on the local Liberty Loan Committee. His last public service was as Chairman of the Committee that effected a revision of the rules of practice of the Orphans' Court. Although he had never held an elective office, he was active in politics at campaign times, and in 1904 was a delegate to the Chicago Convention that nominated Theodore Roosevelt for President.

Besides being an able lawyer and a public-spirited citizen, Clark Olds was an honest, fearless, upright man of the highest ideals. His whole-hearted, sympathetic nature readily responded to every worthy undertaking, and his sterling qualities, with the advantage of a pleasing personality, made and held a host of friends. The late Alfred Noble, Past-President, Am. Soc. C. E., his classmate and associate on the Lake Survey, reciprocated a firm friendship with Mr. Olds and held him in lifelong esteem.

Mr. Olds was elected an Associate Member of the American Society of Civil Engineers on March 1, 1899.

GAYLORD D. WEEKS, Assoc. M. Am. Soc. C. E.*

DIED MAY 6, 1924.

Gaylord D. Weeks was born at Mechanicsville, Iowa, on November 18, 1878. He prepared for college in the Denison, Iowa, High School, and entered the University of Iowa in 1894, from which he received the degree of Bachelor of Science in Civil Engineering in 1898.

After leaving college, Mr. Weeks was engaged during 1898 as City Engineer of Denison, Iowa. In 1899 and 1900, he served as Rodman, Instrumentman, and Timber Inspector on construction work on the Chicago Northwestern Railway.

From September, 1900, to June, 1902, he was Instructor in Civil Engineering and Mechanical Drawing in the State University of Iowa. From the fall of 1902 to April, 1905, he served as Office Engineer in the office of the Assistant Chief Engineer of the St. Louis and San Francisco Railroad Company, at Springfield, Mo., in charge of the Drafting Office, in which position he obtained considerable experience in the design of railroad yards and structures.

In April, 1905, Mr. Weeks became a member of the contracting firm of Hedges-Weeks Construction Company, and continued with this firm until his death, having had charge of the construction of many important railroad and

* Memoir prepared by H. B. Barry, Prin. Asst. Engr., St. L. S. F. Ry., St. Louis, Mo.

other concrete structures in various parts of the Middle West. Mr. Weeks was also a Director of the Jarrett-Richardson Paving Company and the Jarrett Construction Company.

On December 31, 1900, Mr. Weeks was married to Miss Edith Wygath, of Denison, Iowa, who, with two children, Lucille and Gaylord, survives him.

His life was filled with fondness and devotion for his family. He was a man with great strength of character which, together with his fairness and integrity, caused him to be admired by all his personal and business associates. His early railroad experience gave him broad vision, and had much to do with the congenial atmosphere which surrounded all his work as a contractor.

He was a member and Trustee of the First Congregational Church of Springfield, Mo., and was connected with the Masonic Fraternity as a member of Solomon Lodge No. 271 and, also, Abou Ben Adam Shrine.

Mr. Weeks was elected an Associate Member of the American Society of Civil Engineers on June 24, 1914.